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Brigadier A. C. Hughes, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Road Paper No. 39

"Soil Stability Problems in Road Engineering"

by

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SYNOPSIS

The factors controlling the internal stability of earth embankments are discussed, and reference is made to the relation between the degree of compaction and the shear strength of soils. The effect of moisture content on the properties of fine-grained filling and the selection of the moisture content for construction are considered. Embankment design is dealt with, covering choice of profile, materials, drainage, and construction methods.

Embankment foundation problems are examined where peat, very soft clay, moderately soft clay, or complex soil-profiles occur. Bank design and construction methods are discussed, and details are given of methods for improving foundation conditions by removing or strengthening the weak soil.

Slope stability is considered in relation to natural slopes and cuttings. The types of failures are described under both headings, together with appropriate remedial measures. Reference is made to drainage works of several types, the correction of profile, the protection afforded by vegetation, and the insulation provided by coverings of granular materials.

Introduction

Stability problems have been defined as those concerned with safety against the shear failure and erosion of soils in engineering works. In this wide sense the term includes the problem of pavement design, or at least those approaches to it based on the shear strength of the sub-grade, but these have been omitted from the Paper, which is confined to the stability of embankments, cuttings, and natural slopes.

Since excessive settlement of road embankments may cause pavement failure, its cause and prevention are also examined.

Embankments

The Internal Stability of Embankments

Causes of Instability

Since embankments are artificial structures, failure during or after construction must be attributed to either incorrect design or poor workmanship, or to both.

In the past, a common fault in design has been to make the slopes oversteep in cohesive and $c\phi$ soils. That arose from ignorance of the relations between the slope angle, critical height, and the shear strength of the soil. Angles of slope found stable on low banks led, when applied in a rule-of-thumb manner, to higher banks and so to rotational slips.

Where the base of an embankment is normally submerged, it should be designed to withstand rapid draw-down conditions, which arise if the external water level falls so fast that the free water surface in the bank lags behind it. This may cause failure in two ways: by the increase in the unit weight of the fill, from the submerged weight to the fully saturated weight, unaccompanied by any increase in the shear strength of the fill; or by erosion resulting from the escape of water at the toe of the newly exposed face. This effect is most dangerous in fine sand and silty soils. Where a bank is placed under water it is desirable to form at least the base, which cannot be compacted, of free-draining rockfill. In a composite bank this will also act as a drain and, by deflecting the flow lines, help to stabilize the bank under rapid draw-down conditions. (Terzaghi and Peck,* 1948.)

Other faults in design may be inadequate drainage to the top of the bank, or to its foundation; lack of provision for maintaining drains; and failure to protect the slopes against erosion.

With regard to workmanship, the common faults are: inadequate compaction, which may be caused by the use of plant too light for the type of filling, insufficient compactive work, and the careless placing of

* The references are given (in alphabetical order of Authors) on p. 251.

fill in layers so thick that compaction is impossible ; failure to control moisture content during compaction ; poor drainage of the top of the bank during construction ; the placing of fill during very rainy weather ; and the use of compressible soils such as organic clay and peat. Poorly compacted fill has low shear strength and a tendency to soften after placing, and may be subject to long and continued consolidation settlements.

All these faults can be prevented by proper construction control, including site measurements of the degree of compaction in the bank (B.S. 1377, 1948), and by the control of drainage and good organization of the borrow-pits.

The Relation between the Compaction and the Shear Strength of Fill

Shear strength and unit weight are the soil properties which, above all others, determine the stability conditions of an embankment, and since both depend upon the degree to which the soil is compacted, they can to some extent be controlled. Before designing a high bank, tests should be made to establish the relations between these three factors so as to arrive at values which can be used in stability calculations.

The shear strength of the soil in a bank will be governed by :—

(i) The nature of the soil, which is used in the disturbed state.

(ii) The degree of compaction attained in placing.

(iii) The subsequent history of the soil within the bank, which will depend upon its own physical properties, its access to water, and on the conditions of restraint.

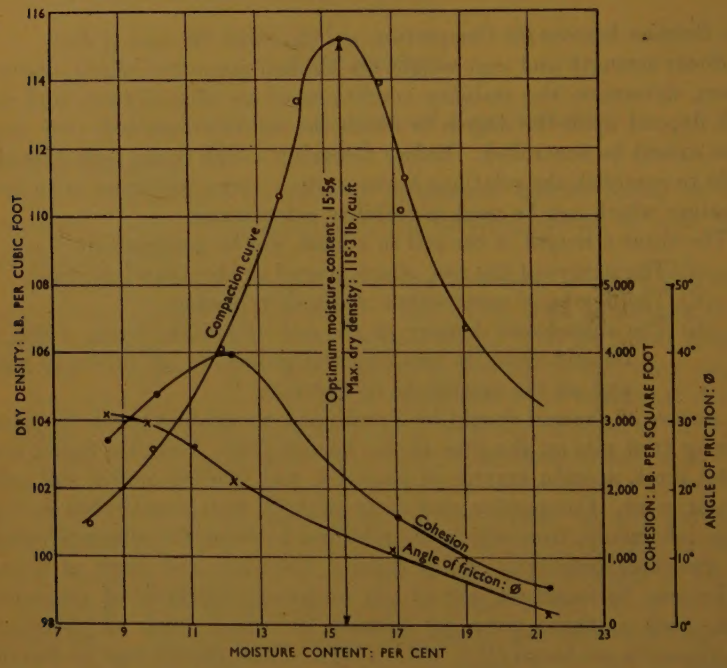
The nature of the soil should be determined on samples taken from boreholes or trial pits on the sites of the borrow-pits. Tests for liquid limit, plastic limit, specific gravity of particles, and grading curves will suffice in most cases. Compaction and shear-strength tests should then be made in the laboratory, from which the relations between the moisture content and the three properties of dry density, cohesion, and angle of internal friction can be found for one of the empirical standards of compactive effort, such as those given by Proctor, on whose work all subsequent developments are based (B.S. 1377, 1948). These results may be recorded graphically (*Fig. 1*).

In the laboratory of the Authors' firm the method used for fine-grained soils is to compact them in the standard Proctor mould, and to take $1\frac{1}{2}$-inch-diameter samples from it by means of the Nixon Extruder (*Fig. 2*). Shear-strength measurements are then made on these samples in the triaxial apparatus by the undrained test.

For coarse-grained soils the particles of more than $\frac{3}{4}$ inch diameter are sieved out and the moisture-content/dry-density relation for the remaining material found in the usual way. Shear tests are then made on soil of known moisture content, and compacted to a known density in a large 12-inch-by-12-inch shear box (*Fig. 3*). Tests under three different vertical loads are made for each of several moisture-content values.

Various workers have demonstrated with different types of soil that at constant moisture content the strength, measured either directly as shear strength or as unconfined compressive strength, increases with dry density. Also, if a soil remains virtually saturated, strength increases with dry density, but is accompanied by a reduction in moisture content. Such conditions are reproduced when soil is compacted wetter than the optimum moisture content. Typical results for saturated soils, derived from such work, are given in Table 1.

Fig. 1

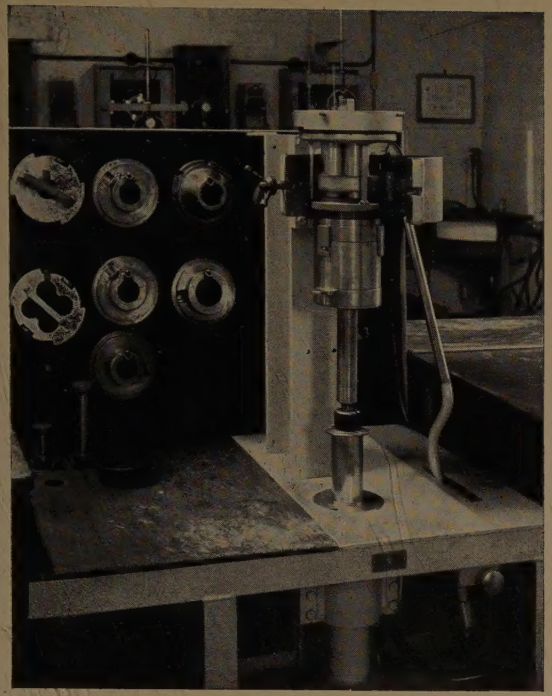


Compaction/Shear-Strength Characteristics (after Golder)

The limited data available from tests on crushed rock and soft chalk are insufficient to enable any corresponding values to be quoted for these materials.

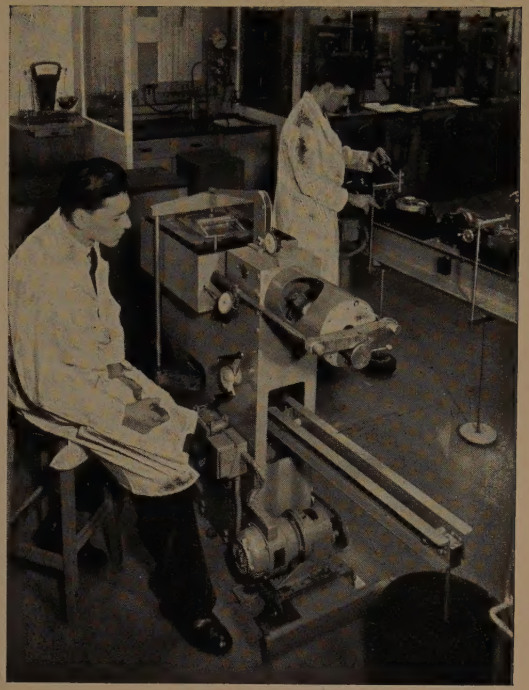
British practice in the laboratory measurement of moisture/density relations is limited to soils of less than ¾-inch particle-size, but in the United States methods have been developed in connexion with the design of high earth dams for working on soils of much larger particle-size. Techniques have also been devised for measuring the shear, consolidation, and permeability properties of coarse materials up to 3-inch size (Walker and Holtz, 1951 ; Hall, 1950). Whether the use of such expensive appara-

Fig. 2



TAKING TRIAXIAL SAMPLES FROM THE PROCTOR CYLINDER BY MEANS OF THE NIXON EXTRUDER

Fig. 3



LARGE SHEAR BOX DESIGNED BY A. W. BISHOP AND D. J. HENKEL

TABLE 1—RELATIONS BETWEEN DRY DENSITY AND STRENGTH FOR SATURATED SOILS

Soil type	Dry density: lb. per cu. ft.	Shear strength		Unconfined compressive strength: lb. per sq. ft.	Authority
		c: lb. per sq. ft.	ϕ: degrees		
Thames Valley Gravel	104*	—	36·5	—	Bishop, 1948 (*a*)
	122*	—	49·5	—	
Sand	90	—	33	—	Golder, 1946
	100	—	40	—	
Sandy Clay	105	500	2	—	Golder, 1946
	115	2,200	13	—	
Sandy Clay	102	—	—	600	Maclean, 1946
	114	—	—	5,500	
London Clay	91	430	—	—	Markwick, 1944
	97	2,800	—	—	

* Calculated assuming specific gravity equal to 265.

tus is worth while in relation to road embankments is open to doubt for the following reasons.

The accurate prediction of the value and distribution of shear strength in a bank long after construction is almost impossible from laboratory tests, though useful information can be gained by consolidated-undrained tests on soil which has been allowed to soften under controlled conditions (Little, 1948; Golder and Ward, 1950; Skempton and Bishop, 1950).

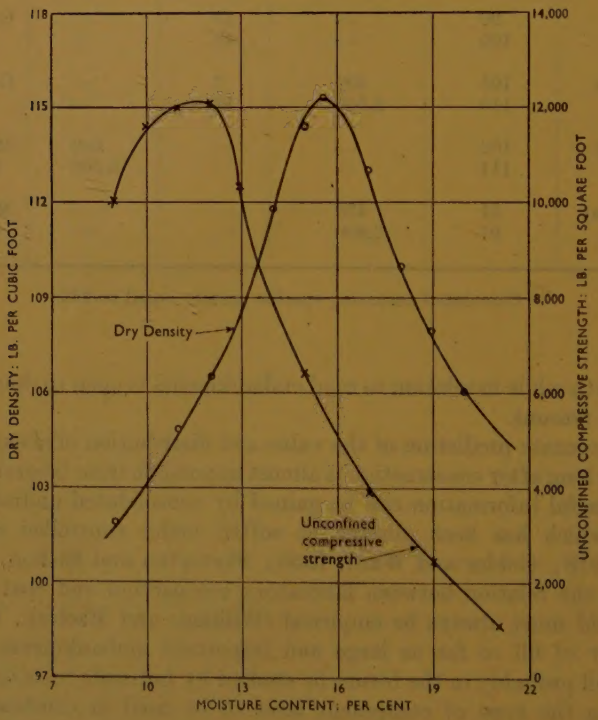
Since the relation between laboratory compaction and that obtained in the field must always be empirical (Williams and Maclean, 1950), the behaviour of fill so far as large and important embankments are concerned will probably in the future be studied by full-scale tests in the field, employing the type of equipment later to be used in construction, for building a trial bank. In any well-compacted embankment there will be a depth below which the overburden pressure will prevent appreciable swelling and softening of the soil. The height of a trial bank must exceed this depth.

Detailed records of soil properties are kept while building the trial bank, and observations of settlement and other bank movements are made during a period of about 12 months. After this time, samples should, if the type of filling permits, be taken from auger holes for further tests, to establish the extent to which softening and consolidation have progressed.

The Control of Moisture Content during Compaction

The maintenance of the moisture content of soil at about the optimum value for normal methods of compaction is difficult in the British Isles throughout much of the year. It is unimportant in coarse granular fill, but if high shear strength and low consolidation settlements are required of a fine-grained soil fill, the moisture content at which it is compacted cannot be ignored. It can be seen (*Figs 1* and *4*) that for maximum shear

Fig. 4



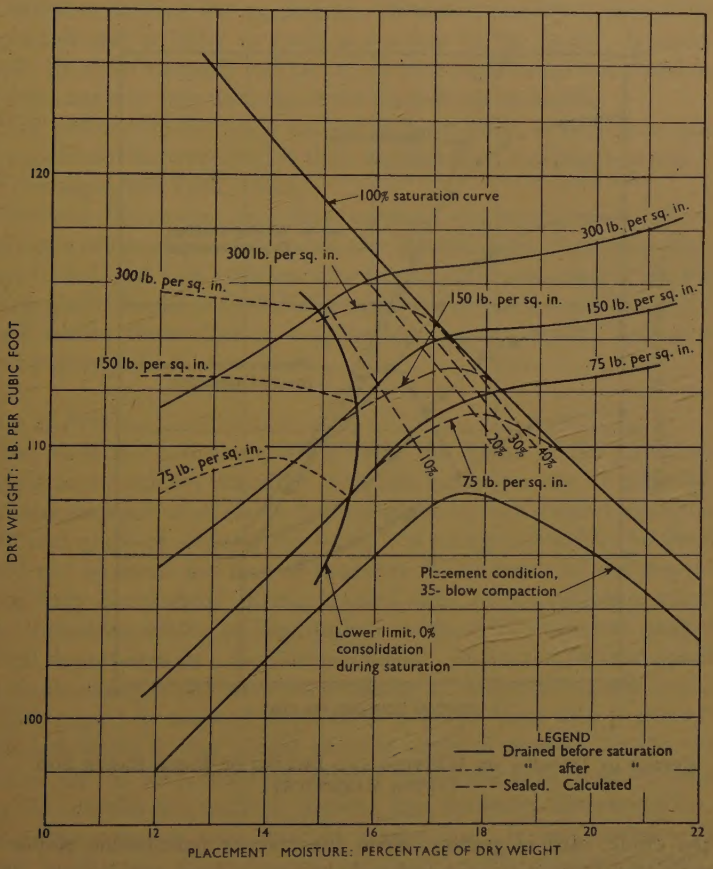
Curves Showing Moisture/Density and Moisture/Unconfined Compressive Strength Relationships

strength such a soil should be compacted at somewhat less than the optimum moisture content. It has also been shown (Holtz, 1948), that if the soil is compacted at a little less than the optimum moisture content the effect of subsequent saturation in increasing consolidation settlement is a minimum (*Fig. 5*). Against this, a soil compacted slightly on the wet side of the optimum value will, when saturated, swell and soften less than one compacted on the dry side of it (*Fig. 6*), but the maximum excess pore-

water pressure attained during consolidation may be greater. Where such a danger exists, pore-water pressure gauges should be installed, and on a sudden rise in pressure, bank building stopped until it has dispersed.

On the whole it can be said that to develop and maintain high shear strength and to minimize consolidation settlement, soil fills should be compacted at moisture contents a little less than the optimum values. As a

Fig. 5



Limiting Moisture Curves (after Walker and Holtz)

general guide to good practice, the air voids should not exceed about 5 per cent, but the actual value will depend on the type of soil and the purpose for which the bank is required.

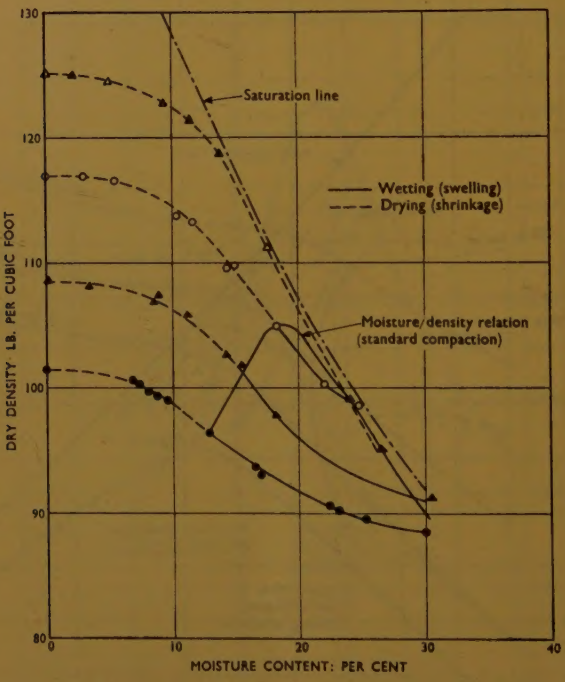
The secret of good bank building lies in site control, both at the

borrow-pits and on the bank, and constant checks should be made on the quality of soil, lift thickness, moisture content, and dry density.

Choice of Material

Since the choice of materials is largely governed by economy, ease of excavation and length of haul may be decisive. For high banks the use of material of high potential shear strength may be justified, even if the

Fig. 6



Effect of Density on Wetting and Drying of Brick-Earth Soil (after Markwick)

cost per cubic yard is greater, since the steeper permissible slopes will reduce the total quantity required and the cost of land acquisition. High-grade fill may be made necessary by local conditions of rapid draw-down, wave action, etc., and where the construction programme and use of a high-quality pavement make long-term settlement intolerable.

Sound rockfill is free-draining and has a high angle of repose up to 45 degrees, but the cost of excavation is high.

Soil filling can be classified as (i) non-cohesive, (ii) $c\phi$, and (iii) cohesive

fill. Soft rocks may be intermediate between (i) and (ii), since they are placed as a granular fill, but by the action of heavy compacting equipment can be crushed into a homogeneous mass. Laboratory tests, however, have shown soft chalk to possess either cohesive or $c\phi$ characteristics.

Sandy gravels and sandfill are readily compacted, but are subject to erosion and wave action. Fine sand will not drain rapidly and if placed at reasonably steep slope angles is unstable under rapid draw-down conditions.

Clay is the least suitable material for filling since it is difficult to control the moisture content during placing. If placed too dry it cannot be well compacted and is likely to swell and soften in the bank. If moist and plastic, its shear strength will be so low that high banks can stand only if the slope angle is very flat, and it will be difficult to handle.

Stiff preconsolidated clays are particularly likely to swell after placing, and should not be employed if their coefficient of swelling is greater than 0·07 (Terzaghi and Peck, 1948).

The best fills possess high shear strength and low compressibility, and are formed of materials well graded over the full range of particle size, from coarse to fine, and with sufficient clay to give them some degree of cohesion.

Composite banks may include material of poorer quality if it is so placed within them that it is protected against erosion, and is at no time overstressed.

Choice of Profile

The elevation and grade being fixed, the design of the profile should be such as to achieve stability with the minimum use of material and minimum occupation of land—factors which in turn depend upon the top width and angle of slope of the bank. In fixing top width the shoulders should not be made too narrow since the fill adjacent to the slopes may not be fully compacted. Erosion, small slips, or consolidation may then cause transverse tilting or longitudinal cracking of a concrete pavement; or local depressions in a flexible pavement. Moreover, reinstatement will be difficult owing to the inherent weakness of the outside of the bank.

With regard to angle of slope, there is no problem with frictional soils which will stand to any height at their angle of repose. In the case of cohesive and $c\phi$ soils, stability against rotational slips must be checked, using the circular-arc method, making any necessary allowances for particular circumstances such as rapid draw-down. In this analysis, values of unit weight and shear strength obtained by the methods already described are used. A factor of safety of at least 1·5 should be allowed in the worst case for large banks, although in some cases circumstances may dictate that lower values be adopted.

If high banks must be built of clay, the safe slopes will be very flat, and it may be found more economical to introduce berms. If these berms are sufficiently wide the slope can be divided into a series of sections, each

of which should be analysed separately, and it may be possible by steepening them to reduce the base width of the bank below that needed for a uniform slope.

Embankments on sidelong ground are liable to move bodily down the slope, and it is advisable when the dip exceeds about 5 degrees to bench the foundation before the bank filling is placed. By providing virtually horizontal terraces beneath the bank a positive key is developed between the fill material and the foundation, and the tendency to slide is eliminated. The number and depth of the benches depend on the slope of the ground, the profile being designed to give the maximum effect with the minimum of excavation.

A final point affecting the profile of an embankment is that it should always be built to its ultimate dimensions, allowing, for example, for the development from single carriageway to dual carriageway. If a bank is widened appreciably, the fresh filling induces additional stresses beneath the existing portion, and causes further consolidation of the foundation so that settlement is increased. Also there will be a tendency for the side of the older portion, against which the new filling is placed, to settle more than the outer side, and for differential settlement to occur between the new and old filling. Renewed movements could easily result in damage to existing pavings or to structures in contact with the filling. Similarly, if the height of an embankment is increased sometime after the initial part has been constructed, further movements will result because additional settlement caused by consolidation is unavoidable if the foundation is compressible. In this case, only structures adjacent to the embankment would suffer. Where cohesive soils are used for the bank material, rotational slips may occur if the slopes were not designed originally to allow for raising the level of the bank.

Drainage of Embankments

Allowance must be made for draining the top, the slopes, and the foundation of an embankment.

On level ground, foundation drains may not be required, but on sidelong ground interceptor drains, consisting of stone-filled trenches carrying open-jointed pipes, should be installed along the high side of the embankment to prevent the accumulation of water in the hollow between the foundation and the bank slope. Deeper drains of the same type can also be used to cut off the seepage from permeable strata where its natural outlet is near or under the fill. All such drains should discharge clear from the bank, but in order to obtain a suitable outfall it may be necessary to provide close-jointed pipes through the fill. If the quantity of water to be dealt with is likely to be large a culvert could be substituted for the transverse pipes.

The surface run-off from road pavements should not be allowed free flow across the verges and down the slopes, or gullies will form. The water

should be collected as on the remainder of the road, and conducted to the bottoms of the slopes in pipes or open channels feeding into the main disposal system. The run-off from verges is small, and not likely to cause much damage where the slopes bear a healthy growth of vegetation.

Slope-drainage systems are rarely employed on embankments until some signs of surface failure are apparent. Their function is to intercept water flowing down the bank and thereby to minimize the amount of softening and erosion. They consist of series of shallow stone-filled trench drains, preferably piped, inclined at about 45 degrees in a herring-bone pattern and feeding into collecting channels running directly up the bank. These discharge into the main close-jointed pipe drain at the toe of the slope.

Berms should not be allowed to collect water running down from the higher benches and a toe drain should be installed along the junction between the berm and the upper slope, connecting with the main drain at the foot of the embankment. Ditches at the bottom of a bank and parallel to it should be kept at a distance of 3 or 4 yards from the toe.

Construction Methods

The stability of an embankment may be influenced both by the method of placing and the method of compaction.

The essential features of placing are that the layers should be uniform and so thin that the specified density can be obtained at the bottom of the lift. For normal compaction equipment the lift thickness should not be more than about 8 inches. Large lumps of fill should not be used if such oversize material is too strong to be crushed by the compacting plant.

The difficulty but desirability of controlling the moisture content of filling during compaction has already been mentioned.

Under favourable conditions a fair degree of compaction is given by the construction equipment itself, particularly by heavy pneumatic-tired units. For high degrees of compaction many types of rollers are in use such as smooth rollers, sheepsfoot rollers, and pneumatic-tired rollers, and in recent years vibrators and vibrating rollers have been developed. The type, weight, and dimensions of equipment employed will depend upon the type of filling, the degree of compaction required, and on the size and strength of the lumps of soil which must be broken down. Probably the most difficult type of fill to compact is stiff clay, for this is lumpy when excavated, but unless thoroughly compacted it is likely to swell and soften in the bank. Heavy sheepsfoot rollers are effective with such material.

Drainage of the bank during construction is essential, and the top surface should be kept with a slight outward slope to allow a rapid run-off and prevent ponding.

Treatment of Finished Slopes

Except for rockfill embankments, all slopes will require protection from erosion, as in the case of cuttings (*q.v.*).

Remedial Measures

Failures of banks (other than foundation failures) which occur long after construction, are generally caused by bad drainage, or the softening of poorly compacted fill. The most usual remedies are counterfort drains, berms at the toe of the bank, or excavation and replacement of the slipped material by lightweight fill.

Cement grouting has been used (A.R.E.A., 1950) to check the mass movements of embankments down sidelong ground, and to stabilize small slips above the toe. The grout is said to increase the bond across the plane of failure. This method is most effective with granular fill.

Foundation Problems

INTRODUCTION

Weak soils beneath a bank may fail by rotational shear during construction, and compressible soils will continue to settle by consolidation long after the bank is built. Such soil profiles have been classified according to their possible modes of failure, as follows (Terzaghi and Peck, 1948) :—

Marsh deposits, that is, peat and very soft organic silt and clay.
Great depths of soft homogeneous clay.
Complex soil profiles.

BANKS FOUNDED ON MARSH DEPOSITS

Properties of Peat

The mechanical properties of peat are influenced by the type of vegetation from which it is formed, by the mineral content, and by the degree of humification. The mineral content is derived partly from the decay of vegetable matter and partly from the addition of water-borne sediment, so that fen peats are always richer in it than moor peats, and, as a class, pass without a break into organic clays. Humification is the process which gradually converts dead plant matter into a black cheesy substance without recognizable structure. It is produced mainly by the action of bacteria and occurs only under water-logged anaerobic conditions.

Peat is characterized by its low density, very high porosity, low shear strength, and by the fact that it is very compressible ; but though its properties differ widely in degree from those of inorganic soils, yet they resemble them sufficiently in kind that the standard soil tests can be applied. For example, Ward (1948 (*a*)) found reasonable agreement between laboratory shear-strength measurements and the shear strength calculated from the $\phi = 0$ analysis of a rotational slip in peat.

The geotechnical properties of peat have recently been described by Hanrahan (1952). In general, the natural moisture content amounts to several hundreds per cent, and since the gas content is large the relation

between moisture content and voids ratio is variable. The specific gravity of the solid matter ranges from 1·2 to 1·7, the wet density from 60 to 63 lb. per cubic foot, and the dry density from 4·0 to 7·5 lb. per cubic foot. Vane tests (A.O.R.G., 1950) have shown the shear strength of peat from different localities to be similar, and in the case of moor peat to range from 3 lb. per square inch at the surface to 1 lb. per square inch at and below a depth of 20 inches. Owing to the abundance of the rhizomes and roots of reeds, fen peats are about 1 lb. per square inch stronger at corresponding depths.

Oedometer tests show that small loads produce large reductions in voids, and that the compressibility decreases with increasing load. The coefficient of volume compressibility for pressures of $\frac{1}{8}$ to $\frac{1}{2}$ ton per square foot ranges from 1·1 to 1·5 square foot per ton. On unloading there is little increase in voids and the material remains almost completely compressed. On reloading, the voids reduction is negligible if the original load is not exceeded (Hanrahan, 1952).

Methods of Embankment Construction on Marsh Deposits

Since marsh soils are so weak and compressible, even low embankments are likely to fail by sinking if the shearing resistance of the dried surface-layer is overcome. Where the depth of soft soil is great, as in the Thames Estuary, settlement under the overburden pressure may continue at the rate of about 1 inch per year, irrespective of any structural load. This movement gives an uneven surface to roads, and may cause cracking in the neighbourhood of manholes, bridge abutments, and other comparatively deep structures. On such soils the following expedients are used :

Removal of the weak soil :—

(1) By excavation.
(2) By displacement :
  (*a*) symmetrical loading ;
  (*b*) asymmetrical loading.
(3) By bog blasting.
(4) By jetting.

Strengthening the soil by consolidation :—

(5) By controlling the rate of building and by overloading ; with or without fascines.
(6) By accelerated consolidation :
  (*a*) with sand drains ;
  (*b*) with cardboard wicks ;
  (*c*) with porous concrete drains.

The use of pile foundations.
The use of lightweight fill.

(1) *Excavation.*—Soft soil underlain by a firm stratum should be removed altogether by normal methods of excavation if its depth is not great, say between 5 and 10 feet, and if disposal areas are available and the cost of fill is low.

Recently the removal of peat by excavation has been used on a very large scale on the construction of a new road between Amsterdam and Utrecht (Ringeling and Vermeulen, 1952).

(2) *Displacement.*—Where the depth of soft soil is great the cost of excavation becomes excessive, and the method of displacement may be used. The bank is then built rapidly so as to induce shear failure in the soil, which is thrust downwards and sideways by the filling. The operation should be planned so as to form a body of submerged material of such shape that it is inherently stable and does not depend to any great extent on the fluid pressure of the surrounding soil for support. If, however, the underground boundaries of the fill are vertical, or worse still, inclined inwards, " weak shoulders " will develop and slope failures may follow. (Dibbits, 1948.)

More recently two methods have been evolved to ensure stability, and each has its place in practice.

(*a*) Symmetrical Displacement.

A shallow trench is excavated along the centre-line of the proposed bank, the stronger crust being removed down to the groundwater level. Fill is then placed in this trench. This method prevents the fill from being supported temporarily on the surface crust and then breaking through on one side only. (*Fig. 7* (*a*)).

(*b*) Asymmetrical Displacement.

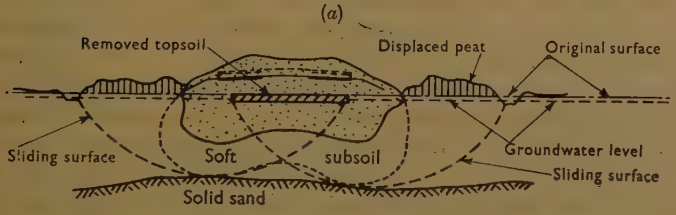
In this method a trench, which may be about 12 feet deep, is excavated along one side of the road line. As filling proceeds the soft soil is pushed up along the line of the bank and is removed by normal methods of excavation (*Fig. 7* (*b*)). The method is more costly than symmetrical filling, but is justified in the neighbourhood of existing buildings. Filling then starts near the building and proceeds away from it.

(3) *Bog blasting* (*displacement aided by explosive*).—The displacement of soft soil can be hastened and made more certain by the explosion of charges buried beneath or ahead of the advancing fill, a method known as " Bog Blasting." A description of this method, with much useful data, has been described before the Road Engineering Division of the Institution (Duncan, Dalzell, and Williams, 1951). Several techniques are in use. In " Toe Blasting " small charges are buried ahead of and parallel to the leading toe of the bank. On blasting a wave of peat is pushed ahead and the toe of the bank settles in behind. The advancing wave of peat can be

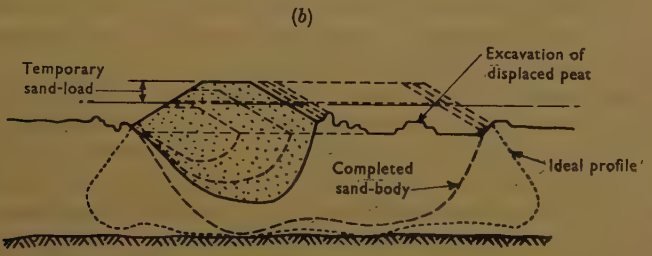
deflected to the side by the judicious placing of fill and the disposition of the charges. This method has been used on a large scale in the United States. (Anon., 1949.)

In " Underfill Blasting," which is used when the peat is at least 10 feet deep, and has been employed on peat beds up to 72 feet deep, heavy charges are buried beneath the bank; at the moment of firing the bank is lifted up, and on falling back the peat is pushed out from beneath it.

Figs 7



Displacement of Peat by the Symmetrical Method (after Dibbits)



Displacement of Peat by the Asymmetrical Method (after Dibbits)

A notable example of this method was its use in the construction of a road embankment 200 metres long across a lake in Sweden (Rudhagen, 1944). Fill was placed first to an excess height of 2·2 metres, and 177 steel pipes, 4 inches diameter, fitted with pointed shoes and a ring to reduce friction, were driven at 4-metre centres, in three rows 4 metres apart. The total charge was 6,148 kilograms (13,556 lb.) of dynamite, the charge per hole in the centre row being more than 40 kilograms (88 lb.), and in the outer rows 22 kilograms (48½ lb.) per hole. At the moment of explosion the bank rose slightly, and then settled 4 metres. Subsequent consolidation settlements have been negligible.

In " Trench Blasting " the charges are placed ahead of the bank in such a way as to weaken the peat and form a trench into which the fill is placed. The method somewhat resembles that of " Asymmetrical Displacement."

It is said that, for peat beds from 5 to 20 feet deep, bog blasting is 25

per cent less costly than normal excavation. Trench blasting is recommended for depths in the range zero to 15 feet, and underfill blasting for greater depths. (Duncan, Dalzell, and Williams, *op. cit.*)

(4) *Displacement by jetting.*—Powerful water jets have been used to displace peat from beneath an embankment, but the method appears to have been superseded by bog blasting. (Kushing and Stokstad, 1935.)

Strengthening the Foundation Soil

(5) *Consolidation.*—The fact that peat, soft clays, and silts are highly compressible and that their consolidation under load is accompanied by an increase in shear strength, is of practical importance, for, by controlling the rate of bank building so as not to exceed a safe load on the foundation during construction, the value of the final safe bearing capacity can be increased. This method is often combined with that of overloading, in which the bank is built to a greater height than is finally necessary, and the foundation soil allowed to consolidate under this excess load, which is removed when comparative equilibrium is reached, after which the road base is constructed. This overloading greatly reduces the long-term settlement caused by consolidation after the road has been put into use.

Over-rapid building may cause local failures, and this should be prevented by control based on pore-water pressure measurements, and to some extent by the use of fascines.

A mattress of fascines beneath an embankment is in Swedish practice regarded as a temporary construction expedient rather than as a permanent part of the structure. Its effect is to act as reinforcement at the base, thus spreading the load and preventing local shear failures, until consolidation enables the soil to bear the full weight. If laid in the dry they will rot in less than 5 years, and the consequent settlements are then made up with more fill. The choice of materials for fascines and their structure need not be described here since an account of their use has recently been published (Civil Engineering Code of Practice No. 2, 1951). The theory of fascines has been discussed by Kjellman (1940), who points out that they have no resistance to bending, but that under a vertical load they have some tensile strength owing to the large area of friction between the branches. If the width of the base of the bank is more than 1·5 times the depth of the soft stratum, the tension in the fascines where cut by the most dangerous slip circle gives a turning moment which acts against the tendency to slip. This effect may reduce the cohesion required for stability, but not by more than 12 per cent. If the width of bank base is less than 1·5 times the depth of soft soil the effect of fascines is negligible, for the slip circle will then intersect them near the toe of the embankment where their tensional strength is low, though some advantage may be gained by extending the fascines beyond the toe of the slope and loading them with a berm.

In Dutch practice fascines form a more permanent foundation and to prevent decay are placed at such a level that settlement will carry them

below ground-water in 6 months (Dibbits, 1948). The sandfill embankment is overloaded to hasten consolidation and when the time/settlement curve shows signs of flattening out the excess fill is removed and a temporary road base of a flexible nature—often of brick paving—is laid.

(6) *Accelerated Consolidation*

(*a*) *Sand drains.*—The rate of consolidation of a stratum of clay is dependent upon the load, the permeability of the clay, and the length of the drainage path whereby the water escapes from the region of hydrostatic excess pressure. In the case of a bank founded on fascines resting on a homogeneous layer the water can escape only to the surface, or to any permeable stratum below the consolidating layer, and the process is correspondingly slow. It can be greatly hastened by providing additional channels for the escape of water in the form of sand piles or otherwise. These, if closely spaced, shorten the drainage path, and hence the period of consolidation. The method of accelerated consolidation by means of sand drains was first described at the First International Conference on Soil Mechanics, held at Harvard in 1936 (Porter, 1936). A Paper on the same subject to the Second International Conference (Stanton, 1948 (*a*)) shows how rapidly the process has developed, for this Author mentions twenty-two sites on which the method has been successfully used. It was recently used on a very large scale on the construction of the New Jersey Turnpike, where 20,000,000 cubic yards of soft silt and clay were stabilized by a total of 5,000,000 linear feet of vertical sand drains from 12 to 100 feet deep (Porter and Urquhart, 1952).

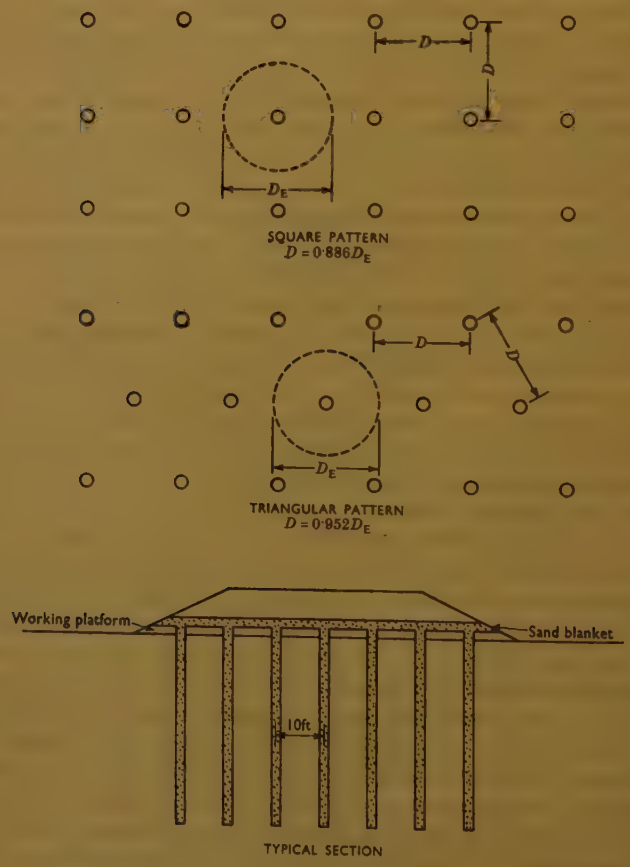
In recent practice the drains are generally about 18 inches in diameter, and spaced at centres equal to from 6 to 9 diameters, on either a square or a triangular pattern. (*Figs 8.*) Drains are installed by driving a steel tube or mandrel fitted with a valve at its lower end, into which sand is poured as it is withdrawn. The tube is generally pitched by means of a crane or excavator fitted with leaders, and is driven with a semi-automatic hammer. Where sand layers occur in the marsh deposits, driving is harder, and jetting has been used, though the disposal of the waste water is troublesome. On very soft ground a shallow fill must be put down first to form a working platform on which the plant can travel. Above this fill a horizontal drain of permeable materials is formed into which the vertical drains discharge, and from which the water may flow to ditches. In some cases the horizontal drain itself is used as a working platform. The sand for both the vertical and horizontal drains must be so graded that it will act as a filter to prevent the escape of silt and yet allow a free flow of water from the ground.

In preparing a scheme it will be required to find the maximum spacing of drains which will give a specified degree of consolidation in a given time for a certain diameter of drain. To make this calculation it is necessary to know the thickness of the consolidating stratum, the coefficient of

consolidation of the clay, and the approximate permeability of the stratum underlying the clay. For a homogeneous clay the method of calculation has been given by Terzaghi (1943 (*a*)) and Barron (1948).

The presence of sand layers should hasten consolidation, but it has been

Figs 8



SAND DRAIN INSTALLATION

shown by Barron that the movement of the mandrel tends to choke the sand layers with clay, a phenomenon which he names "smear." Barron examines the effect of smear, but "in practice the most satisfactory procedure is to carry through the calculations on the simple assumption of homogeneity, and to leave any possible benefits arising from horizontal stratification as a factor of safety." (Skempton, 1952.)

In practice it is usual to check calculations on a preliminary length of bank, and to adjust the procedure in the light of the results obtained.

(*b*) *Draining by cardboard wicks.*—Accelerated consolidation has been developed in a different manner in Sweden (Kjellman, 1948). In this method the drains are formed of cardboard 100 millimetres wide and 3 millimetres thick, made with eleven internal longitudinal channels each of 3 square millimetres cross-sectional area. The cardboard, which is treated with an arsenical bactericide and melamine resin, has a permeability of 0·00001 centimetre per second. Such drains are spaced at about 1·8 metre centres.

The drawback of this method is the high cost of the rather elaborate plant needed to install the drains. At present only one unit appears to be in existence; it was successfully used on the construction of the trans-atlantic airport at Halmsjon, Sweden.

(*c*) *Porous concrete pipes.*—Experiments on the use of porous concrete pipes have recently been described (Saad, 1951). It is claimed that these are more easily installed than sand drains, and are less liable to damage by earth movements during consolidation.

The advantages claimed for accelerated consolidation over the displacement method are that the saving in fill may more than pay for the installation of the drains; the filling can be more thoroughly consolidated; the time of settlement is greatly shortened, and there is less chance of differential settlement caused by pockets of soft ground beneath the fill. Certainly it is now evident that accelerated consolidation has long since emerged from the experimental stage, and is a well-established engineering expedient.

Field control should be exercised when building embankments on marsh soil. When usingthe method of overloading and of accelerated consolidation, constant records of soil movement and of pore-water pressures should be kept. Settlements may be recorded by metal settlement plates, measuring about 3 feet by 3 feet, which are placed on the original ground surface; rods are attached to these and carried upwards to the surface of the fill, through sleeve pipes, which may be extended as filling proceeds. The level of these points should be checked at regular intervals and recorded graphically. Lateral movement at the toe of the bank is a danger signal, and on high fills a row of theodolite stations should be set out near and parallel to the toe, and any movement recorded.

A constant check on the progress of consolidation and of the strength of the soil can be given by pore-water pressure gauges, and these should always be installed, and their readings recorded graphically. Sudden increases in the rate of settlement, lateral movement at the toe of the bank, and sudden increases in pore-water pressure, should all be regarded as danger signals, and if they occur the placing of fill should be held up until the excess pressure has dispersed. Several types of pore-water

pressure gauge are now in use (Terzaghi, 1943 (*b*)) for clay soils; that developed by A. Casagrande (1949) is probably the best.

The choice between excavation, displacement, accelerated consolidation, or any other method, should be guided by thorough site investigation.

Displacement is particularly effective with peat but less so with clay, since with clay, creep and slope settlement go on long after the fill has been placed.

Pile Foundations

Owing to the high cost these are rarely used beneath roads. Cases are described by Dibbits (*op. cit.*) and Alexander (1938), and the theory of piles beneath embankments is discussed by Kjellman (1942).

The Use of Lightweight Fill

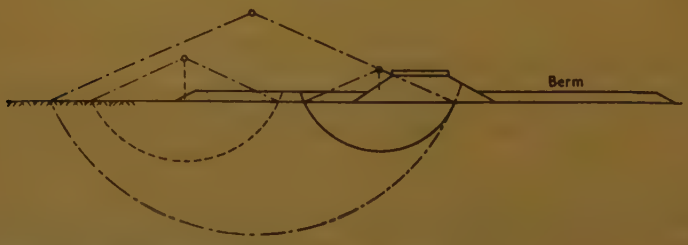
Cinders have been used in bank building so as to reduce the load on the foundation (Gottstein, 1936), and in Holland a case has been described where a road foundation was built of peat which was dried, granulated to $\frac{3}{4}$ inch, and compressed into bales measuring 4 feet by 4 feet by 2 feet (Alexander, 1938).

BANKS FOUNDED ON A GREAT DEPTH OF SOFT CLAY

The stability of the proposed section should be checked by the circular-arc analysis (Golder and Ward, 1950), the most dangerous circle being found by trial and error. The shear strength of the clay should be determined, either by the vane test or by the undrained triaxial test, and it should be remembered that although the $\phi = 0$ assumption gives the correct factor of safety, it does not give the correct position for the worst circle.

If a reasonable factor of safety (say, 1·5) cannot be ensured by flattening the slopes sufficiently, the weight of the bank should be balanced on either side by berms of filled material. This method is generally used on soft recent clays in Sweden (*Fig. 9*) and graphs and tables which greatly

Fig. 9



Possible Failure Conditions for an Embankment Stabilized by Berms (after Kjellman)

simplify the method of design have been published in English (Jakobson, 1948).

If the clay is silty, and hence sufficiently permeable, or if sand layers exist within it, the method of accelerated consolidation may be employed.

BANKS FOUNDED ON COMPLEX STRATA

Banks may fail by sliding out on a thin horizontal stratum of weak soil—either a clay of low shear strength or a sand seam of which the shearing resistance has been reduced by the excess pore-water pressures set up by the weight of the bank. Such cases are described as "failure by spreading," or as "lateral slides."

In designing a bank for these conditions the method of composite surfaces should be used (Cooling and Golder, 1942). It may be found necessary to remove the weak stratum over all or part of the foundation area or to use the method of accelerated consolidation, in which case the rate of bank building should be controlled by pore-water pressure measurements.

It should be remembered that lateral slides may occur on the soft weathered surface of a stiff clay (Bishop, 1948 (*b*)), but the possibility of failure along a thin and unpersistent bed of sand, which might be missed in all but the most careful site investigation, is the more dangerous, and should be borne in mind when designing banks on laminated clays.

The Stability of Slopes

Natural Slopes

Landslips

Landslips are so frequent a cause of damage to roads that their study is an important branch of road engineering, and includes both the analysis and control of existing slips and the recognition of dangerous conditions on the lines of proposed road works.

In recent years, interesting descriptions of landslips have been published, notably those by Knox (1927), Ladd (1935), Sharpe (1938), and Ward (1945), and a fundamental study of their causes by Terzaghi (1950). Many types of earth movement are described and classified by these Authors, some of rare occurrence.

In road engineering two types are important: landslips, which may be defined as superficial earth movements having a horizontal component and characterized by rapid movement and limited extent and duration; and "creep," which is the slow continuous downhill movement of the surface layer on steep natural slopes, under the action of gravity, hastened by seasonal changes. The causes of landslips have been divided into three classes. The first comprises external causes which operate by increasing

the shearing stresses within the slip zone, and include alterations in the height or profile of a slope by natural erosion or by excavation, the effect of loading by dumping fill or waste material at the top of a slope, the steepening of slopes by tectonic movement, and the effect of earth tremors ; and the second, internal causes which lead to failure by reducing the shearing resistance within the slip zone, by increase of pore-water pressure in frictional material, or by the loss of cohesion as, for example, in the swelling and softening of clays. This logical approach is attributable to Terzaghi (*op. cit.*). It has clarified thought on the problem of landslips, and is a most useful approach to particular cases.

To these Terzaghi adds a third and intermediate class which includes internal erosion, rapid draw-down, and the spontaneous liquefaction of dilatant sands. In the same Paper he disposes once and for all of the common error of attributing landslips—since they often occur after heavy rain—to the lubricating effect of water. His arguments on this point will be repeated here, for this false notion is still strongly rooted in the minds of many engineers and geologists. They are : (1) lubrication is essentially a surface-film effect and in all but arid climates soil below the immediate surface layer contains sufficient water at all seasons, and irrespective of recent rainfall, to give full lubrication of the soil particles relative to each other, were such an effect possible, but in fact (2) in the case of many common rock-forming minerals such as quartz, the effect of a film of water is to increase the coefficient of static friction between particles, not to decrease it.

Types of Failure

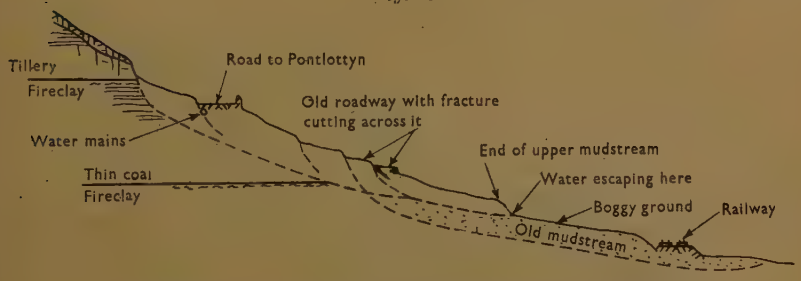
External causes.—Erosion by the sea at the base of a slope steepens it, and by a river may both steepen it and, by valley-cutting, increase its height ; both effects tend to instability, particularly where clay outcrops at the base. For the clay, being overstressed, yields by plastic flow, and the consequent loss of support may cause the collapse of overlying strata of greater strength. This effect may be seen on the cliffs at Folkestone, where the chalk overlies the Gault clay ; in the Island of Skye, where Tertiary Basalt overlies the Lias ; and elsewhere. The resulting slips are in some cases so large that they have been mapped by geologists as faults. It might be supposed that, after a slip, temporary equilibrium being established, it would only be necessary to prevent erosion by normal methods of coastal protection to prevent further movement. Unfortunately this is not always the case, since other processes may be at work, such as periodic variations in artesian pressure or internal erosion (*q.v.*), both of which may lead to progressive failure. On a smaller scale the erosion by streams at the base of slopes of glacial till on valley sides is a frequent cause of damage to roads. Such material generally has some slight cohesion because of the presence of capillary moisture and a small proportion of clay, so that it will withstand undercutting before failure ; the new and

steeper face thus exposed is potentially unstable and will soon fail again. The only cure is to build a retaining wall at the base and to restore the slope with granular fill; but this is costly, so the base of any threatened slope should be protected before failure is imminent.

The effect of excavation is obvious, but it might be repeated that clearing and excavating the toe of a slip is the surest way to cause further movement.

The addition of fill at the top of a clay slope increases the shear stresses within it, and may lead to failure; for example, colliery dumps are said to have been the cause of large landslips in South Wales (Knox, *op. cit.*). Saturation by water, apart from its other effects, increases the weight of a soil mass, and may thus tend to cause failure, as in the case of rapid draw-down. Earthquakes are a frequent cause of landslips, though

Fig. 10



DETRITUS FLOW IN SHALES AND SANDSTONES (AFTER KNOX)

one which is fortunately unknown in the British Isles. The horizontal component of the acceleration due to the shock reinforces the turning moment due to gravity, which tends to cause slope failure.

Internal causes.—In the presence of water and with reduction of pressure, soft clays and some shales swell and soften. Thus the debris of shear failures in shaly strata, particularly if saturated from beneath by springs, form secondary or detritus flows (*Figs 10* and *11*) which can be most troublesome since they continue to move on gentle slopes. As the debris is removed in this way, repeated shear failures above may act as a source of material to the flow. Since such movement is characteristic of other soft schistose rocks it has been suggested that the mechanism of flow is similar to that of dilatant sands (*q.v.*). "During each dry season . . . fragments disintegrate to a certain extent. Under the increased load due to saturation in the following wet season some of the fragments fail and transfer their load to the water." (Terzaghi and Peck, 1948.) Drainage is most effective in checking such flows (Knox, 1927).

In weakly cemented rocks cohesion may be destroyed by the solvent action of water.

The shearing resistance of a soil may be expressed as :

$$S = c + (P - U) \tan \phi$$

where c denotes the cohesion, P the overburden pressure, U the pore-water pressure, and ϕ the angle of internal friction. Should a stratum of sand exist beneath a slope, an increase in the pore-water pressure within it may reduce the frictional strength, by increasing U, to such a degree that failure will occur. It has been demonstrated (Terzaghi, 1950) that the periodical movement of certain large slips is related to changes in artesian pressure in an underlying aquifer, and that this in turn is dependent upon the variation of rainfall over periods of years. It has been suggested that the Folkestone Warren landslips are of this type. Such slips are not uncommon in varve clays. They are characterized by sudden and rapid

Fig. 11

Detritus Flow showing the Effect of Springs (after Knox)

movement, and the great distance of translation on an almost horizontal surface. An interesting example is described and analysed by Odenstad (1951).

A particular case of loss of shear strength is that associated with " Quick Clays," which are extremely sensitive to remoulding. Such a clay has been described which, in the undisturbed state, had an unconfined compressive strength of 0·8 lb. per square inch, and an angle of internal friction of 8½ degrees in the consolidated-undrained test, yet on remoulding " drips from the fingers." (Peck, Ireland, and Fry, 1951). If failure is once initiated in such material by erosion or otherwise, slips may thus occur which flow far on an almost horizontal surface.

Internal erosion.—Springs flowing from an outcrop of fine sand may carry material with them and so form cavities in the moist sand above the inclined ground-water table. These cavities tend to work backwards and increase in size until they eventually lead to the collapse of overlying

strata. A case of progressive slipping attributable in part to this phenomenon has been described by Ward (1948 (*b*)).

An example of what might be described as the internal erosion in clay, which led to serious landslides, has recently been described (Drouhin, Gautier, and Dervieux, 1948). Water derived from calcareous rocks passed through sediments containing glauconite. Here some of the calcium in solution was replaced as a result of base exchange by potassium (Proix-Noé, 1946). The water then entered a fissured clay at the base of the slope, and as a result of further base exchange the clay deflocculated, and appreciable amounts were removed in suspension. This led to further softening and eventually to collapse of the overlying strata.

Dilatant sands.—Saturated sands in a very loose state of packing may behave temporarily as liquids if, by vibration or otherwise, their structure is broken down and the stress transferred from the granular skeleton to the liquid phase. This phenomenon has been the cause of slips (Peck and Kaun, 1948).

Recognition of Dangerous Conditions

Where roads must follow valley sides, all very steep slopes other than those on sound rock should be suspect. Thus glacial till in many valleys is often nearly at its angle of repose, so that excavation may start slips above a road ; similarly on clay soils material dumped on the lower side of the road may promote shear failures. Design by the methods of soil mechanics is useful in such cases.

Troublesome slips in the north of England have occurred because roads have been located across old landslip areas. Old landslips, some of them very large, commonly occur on the sides of valleys in carboniferous shales and sandstones in north Derbyshire and west Yorkshire, and their debris extends in a hummocky mass far down the hillsides. The main slips probably occurred during a period of intense erosion at the end of the last glacial period, and have long been quiescent, but if roads are built across the debris secondary slips are started. Aerial photographs are useful in detecting old slip areas.

Remedial Measures

Diagnosis must precede cure. In the case of a large slip a contour map should be made to a scale of, say 1/600, on which cracks, scarps, and flows of water should be clearly indicated. The geology should also be thoroughly explored by borings or trial pits, and if necessary by pore-water pressure measurements.

Drainage is the most valuable expedient since it reduces pore-water pressure, hastens consolidation, leads to an increase in the shear strength of the soil, checks erosion, and reduces the effect of frost. In fine sand and silt, drains should be provided with suitable gravel filters to check the loss of ground. Before drainage is undertaken the entry of water to an

unstable area should be prevented so far as possible. Leakage from pipelines or other sources should be checked, and any cracks or open fissures drained and sealed. In some cases elaborate measures have been taken to seal the surface of slopes with concrete and asphalt (Anon., 1928), and soil stabilization would also be effective for this purpose.

Small landslips where the unstable ground is comparatively shallow and underlain by firm material may be cured by constructing a rubble-filled interceptor drain trench with a suitable outlet carried down to bedrock right round the back of the slide. This expedient is as a rule too costly for large slides. Counterfort drains have proved most successful in checking flow slides and shallow shear slips (Knox, 1927; Duvivier, 1940). They are rarely used on large landslips owing to the cost of deep trenching.

From the literature (Anon., 1947; Balsille, 1936; Forbes, 1947; Huntting, 1945) it would seem that there are two methods in common use for draining landslip areas by adits. In the first a heading is driven in from the toe of the slide against the direction of movement. If the slide area is large, several parallel headings will be required. This method has several disadvantages; although the length of drive is short, work is in disturbed ground and may be costly and, if the ground is still moving, dangerous. Moreover it will usually be necessary to pipe, and back-fill the heading with rubble if it is to remain effective. The second method is to drive in alongside the slip for some distance and then turn at right angles beneath it. Vertical drains (*q.v.*)—either boreholes or shafts—are then put down through the slip so as to discharge into the heading. This is probably the better method in every way, but work must be preceded by a thorough site investigation.

Since adits are costly, horizontal borings have been used with success in draining landslips in partially decomposed rock (Stanton, 1948 (*b*)). Holes up to 200 feet in length were drilled into the slip at about 20-foot centres, using a 4-inch auger bit tipped with hard metal. After drilling, a perforated casing pipe, 2 inches in diameter, was jacked-in in 20-foot lengths, butt-welded on site. Near the surface, plain pipes were used to prevent the entry of roots. The method is inapplicable if much hard unweathered rock is present.

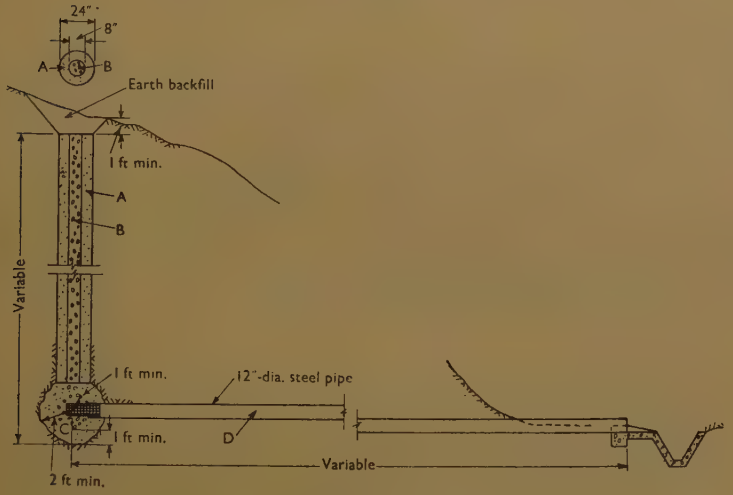
Vertical drains are most useful to reduce pore-water pressures where sand layers are interstratified with clay, as is often the case in glacial deposits, or to drain thick beds of sand subject to internal erosion. Such drains must lead down either to adits, if underlying firm ground exists within reach, or to horizontal borings through which the water is discharged. Two cases can be quoted.

The stabilization of a slope in glacial deposits subject to slips due to hydrostatic pressure in lenses of sand has recently been described (Palmer, Thompson, and Yeomans, 1951). Horizontal zinc-coated steel pipes (D), 12 inches in diameter and fitted at the leading end with a 2-foot length of silicon-bronze well-strainer, were jacked and jetted from the toe of the

slope for distances of 110 to 160 feet. A vertical boring of 24 or 36 inches diameter was sunk by driving and jetting to intersect the end of each horizontal pipe and stopped one foot above it. Excavation was completed by hand to form a cavity 4 feet in diameter at the intersection point (C).

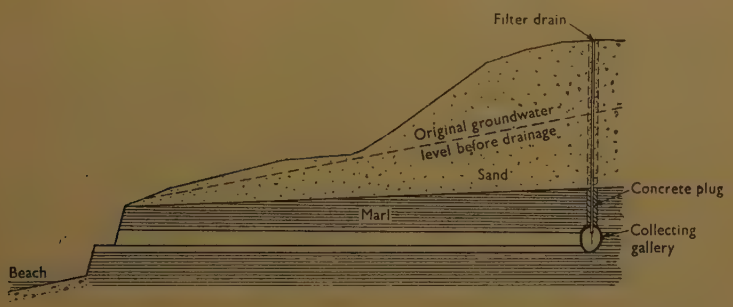
Figs 12

(*a*)



Vertical Filter Drain (after Palmer, Thompson, and Yeomans)

(*b*)



Vertical Filter Drain (after Rodio, etc.)

A two-layer gravel and sand filter (A, B) was then placed in the vertical hole and the casing withdrawn (*Fig. 12* (*a*)).

A rather similar method was used to check slips caused by internal erosion, which were threatening the Route Nationale No. 11 between Algiers and Mostaganem (Colin, 1940). Here a stiff marl outcropping on

the beach was overlain by fine Pliocene sands (*Fig. 12* (*b*)). Graded filter drains, 1·2 metre in diameter, were sunk through the sand into the marl and a discharging pipe was concreted in at the lower end. A concrete-lined gallery was then driven from sea level to pick up all these drains, the water discharging into the sea. In some cases, internal erosion in a bed of fine sand may be checked by placing inverted filters over the points of egress, which allow the water to escape freely without loss of ground (Ward, 1948 (*b*)).

Slope correction and the use of berms.—On small slips, and where the topography permits without excessive quantities of excavation, landslips may be cured by flattening the slope, or by the introduction of berms by excavation in the slope, or by filling at the toe.

Effect of vegetation.—Since much water is removed from the ground by transpiration the cutting down of trees may certainly be a cause of landslips, but there is some controversy as to the effect of afforestation on large slips. It has been stated that it is without effect on stability (Messines, 1944), and even that on clay slopes the trees may be a cause of slips because of their weight (Goguel, 1943 ; Odenstad, 1951).

Observation of Movements

Where roads or railways must cross landslip areas too large to be controlled, it is important to know when danger impends. This is possible by accurate observation, since every slip must be preceded by a period of increase in strain until the point of ultimate failure is reached. In Sweden, electrical devices have been used to detect such premonitory movements.

Cuttings

Deep-Seated Failures

Cuttings in sand and silt.—Since the shear strength of sand increases with overburden pressure, sand slopes below their angle of repose are not subject to deep slips unless their stability is affected by internal erosion or by seepage pressures. In modern practice the excavation of a cutting in sand below the ground-water level will, as a rule, be carried out with the aid of ground-water lowering by well points or filter wells. So long as pumping continues the slopes will remain perfectly stable, but provision must be made to ensure their continued stability after excavation has been completed and the ground-water allowed to rise once more. If the bottom of the cut is not far below the natural water level the usual drain at the toe of the slope will be sufficient, but if the depth is great it will also be necessary to load the toe of the bank with an inverted filter which will allow the water to escape freely to the drain, but will prevent both loss of ground and instability caused by seepage pressure. In the case of deep cuttings, flow-net analysis may prove necessary.

The case of a cutting in silt is very similar, since here again pre-drainage

will generally be used, and loaded sand filters may be necessary to maintain stability (L. Casagrande, 1947).

Cuttings in clay.—In homogeneous and intact clays the conditions of stability are comparatively simple and depend upon the angle of slope, β, the depth factor D_f (or ratio of the vertical depth from the top of the cutting to bedrock to the depth of the cutting), and on the stability number $\frac{S}{\gamma H}$, where S denotes the shear strength of the clay, γ the density of the clay, and H the depth of the cutting. Such cases are readily solved by the published tables and graphs (Taylor, 1937). The possible types of failure thus depend upon the depth of the firm stratum and have been classified accordingly. Where the firm ground is at the base of the cutting a toe circle is the most probable type of failure ; this will emerge at the toe of the slope. If the firm ground is at a shallow depth below the cutting, failure may be along a slope circle, tangential to the top of the firm stratum and emerging above the toe of the slope ; and finally, if the firm ground is at a great depth, base failure may occur along a mid-point circle, which will also be tangential to the firm ground, but whose centre will be located above the mid-point of the slope (*Figs 13*).

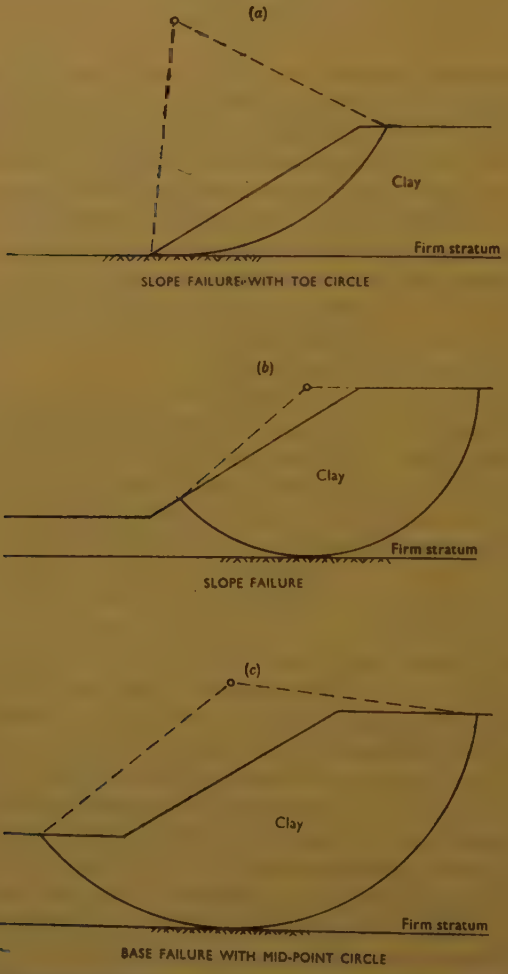
The geologically older clays which have been subjected to pre-consolidation loads are generally of the stiff fissured type, being traversed by a network of joint planes. The excavation of a cutting relieves the stress in the body of the clay and the joints open to form fissures in which water accumulates. The clay then swells and softens into these fissures, and unequal expansion causes further fissures to open. In this way the overall shear strength of the clay is progressively reduced until the slope becomes unstable and a slide occurs. It should be noted that such slides are generally shallow and pass through the toe of the bank, for the clay below excavation level retains its original strength longest.

The London Clay is a typical stiff fissured clay, and Skempton (1948), from the analysis of six failures of slopes and retaining walls, has drawn curves showing the decrease of shearing strength along a surface of failure with time. He concludes that the curves are asymptotic to a value which is presumably the average shear strength of the softened clay in the fissures ; that this softened shear strength is greater at greater depths ; that on shallow cuts softening proceeds more quickly owing to seasonal effects ; and that the strength of the clay at failure is not a particular value which can be used to characterize the clay, but is merely a value corresponding to a factor of safety of unity for the given slope. " This strength will be an average value depending on the final strength, the strength of the unsoftened clay and the proportion of the slip surface which passes through the fully softened clay." Quoting Collin (1846), he points out elsewhere (Skempton, 1949) that ultimately clay slopes will weather back to the slopes exhibited in natural hillsides ; for London Clay the slope is about 1 in 6.

In practice, then, two measures should be considered. The slopes

should be as flat as is consistent with economy, and examination of the examples quoted above suggest that 1 in 3·5 gives a section which should remain stable for many years, whereas slopes of 1 in 2·5 are likely to prove troublesome (Cassel, 1948).

Figs 13



TYPES OF FAILURE IN A BANK OF HOMOGENEOUS CLAY

The surface adjoining the top of the cut should be thoroughly drained for a distance back equal to the depth of the cut and the face of the slopes should be sown down to grass as soon as possible after construction.

Cuttings in peat.—The necessity to excavate a cutting in a raised bog

is rare, but a case has been described (Mills, 1912). A pilot trench was excavated within timber sheet-piling, and the peat was thoroughly drained before the main excavation was started. The increase in strength thus obtained enabled the main excavation to proceed without failure.

Cuttings in complex soil profiles.—Here failure along thin strata of low shear strength may occur as already described in the case of natural slopes.

Remedial measures.—The methods used to stabilize natural slopes are applicable to cuttings, but since in cuttings the height of the slope is comparatively small, the counterfort drain is by far the most usual expedient. One of the Authors of this Paper has presented to the Institution some notes on modern practice (Glossop, 1947).

Surface Failure by Erosion and Frost Heave

Vegetation should be established at once on newly excavated slopes, for it prevents erosion of sands by wind and rain, and checks the drying and cracking of clays. Since for any given soil, erosion is more rapid the steeper the slope, and since rapid erosion makes planting difficult, it may be desirable to make slopes flatter than is necessary to prevent rotational slips. Useful data on the angles of slope in various soils and rocks and their effect on vegetation have been published (Toms, 1948 ; Civil Engineering Code of Practice No. 2, 1951).

In regions of cold climate, and where soils subject to frost heave are found, soil flow may develop on a very large scale from artificial slopes (Lane, 1948). This phenomenon is associated with glacial silts and particularly those showing varved structure. During cold periods frost may penetrate to a depth of, say, 2 or 3 feet, and ice lenses will form in the silty strata, the water being drawn from more permeable sandy beds. With the thaw, melting sets in at the surface, and to a less extent from the underside of the frozen soil. The surface soil thus becomes heavily charged with water from the ice lenses, and the saturated soil flows down the slope in a semi-liquid condition.

Precautions and Remedial Measures

Much useful information on soiling and planting slopes will be found in the Proceedings of the Conference on Biology and Civil Engineering, 1949, but certain points in modern practice may be mentioned here. Where good top-soil is not available sewage sludge can be used, which should be at least 1 year old and have been stacked in heaps not higher than 5 feet. Most weeds, which are annuals, will be eliminated if cut twice during the first 2 years.

The question of grass mixtures is one on which there is much difference of opinion, but Doran (1948) points out that standard specifications are dangerous since much depends upon site conditions, and recommends simple mixtures based on local experience. To protect the grass seed from

erosion until it has germinated and taken root, various methods are employed, such as contour ploughing, the use of longitudinal runs of turf 6 inches high pegged down at 5- or 6-foot centres down the slope (Toms, 1948), or protection with cheese cloth or a thin covering of reeds or straw (Doran, 1948). Such protective covers also conserve the moisture and prevent the loss of heat by evaporation; both effects hasten germination. A recent development is to cover the freshly seeded slope with a thin layer of sand; this is sprayed with a non-toxic bituminous emulsion on which more sand is added. This protects the seed, but can be penetrated by the shoots (Sutton, Alexander, and West).

With regard to fertilizers it is usual to add 4 cwt per acre of superphosphate at the time of seeding, and 1½ cwt per acre of nitro-chalk after the grass has germinated (Moran, 1948). Weed control is necessary to protect farms, and recently selective hormone sprays have been used with success (Boyce, 1951).

Where fire risks are great, alfalfa may be used in place of grass.

In chalk cuttings, ivy planted in pockets of good soil at 6-foot centres and *Clematis vitalba* give protection against frost.

Authorities are agreed that shrubs and trees check minor slips in cuttings by the effect of transpiration and the reinforcing effect of their roots. The most widely used species in Britain are *Cotoneaster henryana*, False Acacia, Mountain Ash, and Birch.

Slopes subject to wave or river action should be protected by stone pitching, faggots, or fascines. Detailed descriptions of such work have recently been published (Dobbie, 1946; Civil Engineering Code of Practice No. 2, 1951).

Soil flow caused by frost heave can be prevented by covering the surface of the slope with from 6 to 12 inches, or in extreme cases, up to 30 inches of permeable material, either sand, or better, cinders being commonly used. The effect of this layer is partly to act as an insulator which checks melting from the upper surface of the frozen soil while permitting it on the lower surface, from which the excess water can escape, and partly by its weight to re-consolidate the moisture-laden surface layer.

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The Paper is accompanied by two photographs and eleven sheets of drawings, from which the half-tone page plate and the Figures in the text have been prepared.

Discussion

Mr B. F. J. Bradbeer observed that it was not easy to follow on after the striking examples presented by Mr Glossop when introducing the Paper.

Instability in roads might be considered, he suggested, under two general headings. The first was the more spectacular failures of earthworks and landslides, such as the case recently reported by the County Surveyor of Worcestershire, which had cost £10,000 to rectify. At present-day prices it was doubtful whether any slip worthy of the name could be dealt with for much less than £5,000. Secondly, there were the innumerable day-to-day instances of local faults caused by the adverse effects upon potentially weak subsoils of wear and tear, incursion of water, severe frosts, and so on. Those minor features went almost unnoticed by the public, because they were dealt with during ordinary maintenance, but the total cost of them was obviously considerable.

The great costs involved in these matters were worthy of continuous study by the local engineering practitioner, with, in cases of grave doubt, the engagement of the specialist engineer to assist in arriving at a proper diagnosis. The Paper should greatly assist the practising engineer and the student to understand the potentialities of the various soils upon which the structures of his roads had to be founded and give him confidence so that he would be able to assess the strengths of those natural materials just as he would calculate the strengths of the artificial materials of which he would construct a bridge or other building. One might even say that the road engineer could not properly fulfil his duties until he was able to recognize the soils, geological formations, water-bearing propensities, and typical verdures of the several regions in his area. Only so would he be able to make the best use of natural materials in new works and be better able to diagnose faults in the course of ordinary maintenance. Much could be learned from the study of old and new Ordnance maps, the geological sheets and the published Memoirs which were available in most regions. The possible significance of ancient place-names should not be neglected, e.g., Clay Hill and Foulride Green.

In East Sussex there was a great deal of chalk which was very stable in cutting and filling if, as the Authors had pointed out, drainage was taken care of and protection afforded against severe frost. In Sussex, however, there was on the whole a great dearth of stable material, and it had always been necessary to deal with several kinds of clay, which covered very large areas. The worst was Gault clay, which could have a liquid limit exceeding 100 per cent and a clay content as high as 60 per cent. That could be badly affected by seasonal changes in moisture content for a depth of as much as 4 feet below a road surface.

Some road surfaces had been observed to alter in level as much as 2 inches between summer and winter. That was not uncommon. Roadside trees appeared to have a marked effect upon such a soil, and roads of total thickness of 24 inches were shown to be inadequate on such a foundation.

There was a large tract of Weald clay which, if it was compacted with proper moisture control, could be made to form stable road beds and embankments. It was not easy to control moisture in earthworks in the British climate, but use had been made of weather forecasts, and the extra cost of maintaining outward-sloping surfaces and stoppage of work in bad weather had been justified by the years of trouble-free life of embankments so dealt with. In one case, which Mr Bradbeer illustrated by means of a lantern-slide, the road upon a high embankment of Weald clay had been in its present satisfactory state for 17 years. The clay was not difficult in cuttings of 1-in-2 slopes, if the banks were quickly soiled and planted, as advised by the Authors. Complex strata dipping towards a road in cutting of such material, however, required special precautions.

In the north and north-east of the county there was a great deal of Wadhurst clay, which was found to be stratified and in flat lenticles at various depths and much broken by faulting. Many of the slips and landslides on old-established roads had taken place on Wadhurst clay because of the incursion of water into the strata from over-lying caps of Tunbridge Wells and Ashdown sands. Those sands were extremely fine, containing as much as 60 per cent of rock flour smaller than 60 microns which, when affected by severe frost, behaved like silt and resulted in frost blows. Those sands were interspersed with layers of semi-indurated sandstone, which could provide channels for the incursion of water. Such fine sands could take up a great quantity of water and hold it for a long time, even on hill-tops, where old maps showed that the greater number of habitations existed on such sands where, before piped supplies had been available, potable water had been obtained from shallow wells in the sand.

The strong retention of water in those sands made the rapid rectification of slips difficult, and indeed dangerous, by ordinary drain trenching methods, and they could not easily be dealt with until the water evaporated in fine weather. For the same reason, drainage by well-points was not feasible. In considering such matters Mr Bradbeer had had much assistance from the Paper [1] presented to the Institution by Messrs Glossop and Skempton some years ago. When water-logged, the shear strength of those sands was very low, and continuous horizontal movements could take place at slopes as flat as 1 in 12.

The Tunbridge Wells and Ashdown sands, when excavated and mingled

[1] Rudolph Glossop and A. W. Skempton, "Particle-Size in Silts and Sands." J. Instn Civ. Engrs, vol. 25, p. 81 (Dec. 1945).

with their banded strata of sandrocks, made excellent filling material of high dry density. Tests on a 30-foot embankment 13 years old had given a California Bearing Ratio of about 30 per cent at a natural moisture content of 15 per cent, and a dry density of 113 lb. per cubic foot. It was considered that that satisfactory result was largely attributable to separate billing in the contract of all compaction work at day-work rates.

As the Authors had pointed out, care was necessary to avoid drawdown if the toes of slopes were likely to be submerged in water, and open ditches had to be kept clear of such slopes. In cuttings in those sands, the slopes had successfully stood for 20 years if not steeper than 1 in 3 and well protected by counterfort drains.

Mr Bradbeer showed a number of lantern-slides illustrating the conditions he described, making acknowledgement to the Director of Road Research for permission to exhibit two of the slides.

Mr D. J. Maclean said that it was now 13 years since road works involving large amounts of earthwork had been carried out on any considerable scale in Great Britain. As a result, there had been very few opportunities for road engineers to gain experience in the practical design and construction of major earthworks. The same period of 13 years had seen a tremendous advance in the application of the principles of soil mechanics to such matters. Although most road engineers had become versed in the new science, it was always valuable to have Papers from people such as the Authors, who had had much practical experience of the many problems which were involved.

It was probably worth mentioning that on any road scheme where major earthworks were involved, their cost might amount to as much as 50 per cent of the total cost of construction. It was therefore clear that if the proposal to construct motor roads in Great Britain was eventually implemented, the cost of those works would depend very much on the quality of the design and construction of the earthworks.

Before dealing with detailed points in the Paper, he would like to make one general comment. The Authors had aimed at covering in a very comprehensive way all the problems that were liable to arise in the design and construction of earthworks, and that had necessarily meant that some matters had to be dealt with in a somewhat cursory way. His personal preference would have been for a Paper which limited itself to a more detailed discussion of the more important problems.

His detailed comments would be mainly concerned with embankments. The Authors appeared to regard stability of slopes of high embankments as the principal design problem. The term " high " had not been defined, but, if it meant embankments higher than 30 feet, he would think that such embankments were unlikely to be common in Great Britain, and in any case were to be avoided where possible by suitable adjustment of the road alignment. His view was that in Britain it was the comparatively low embankment which was the main concern of the road engineer, and

in such cases the principal design problem was to prevent the occurrence of differential settlement, which could be so detrimental to the performance of the carriageway.

In considering the design of the slopes of high embankments, the Authors rightly pointed out that in view of the difficulty of estimating the ultimate moisture content and state of compaction that would be reached by the soil in an embankment it was impossible at present to make an accurate design of the slopes, and it was therefore necessary to use factors of safety as high as 1·5. That was tantamount to saying that very flat slopes would have to be used in such cases. The high cost of doing that seemed to be a good reason for avoiding very high embankments wherever possible.

In the section of the Paper dealing with the control of moisture content during compaction, the Authors had concluded that the soil should be compacted at a moisture content a little less than the optimum value, presumably that obtained by the British Standard compaction test. It was a little difficult to reconcile that with the opinion also expressed by the Authors, that soils were liable to increase in moisture content after construction. If that were the case, would not it be better to compact at the moisture content which, it was estimated, would ultimately be reached by a soil? In the absence of reliable information, however, it was a little difficult to say what changes did occur in an embankment after it was constructed, and in any case road engineers would usually compact soils at the moisture content at which they found them during the construction period, although they would do what was practically possible to prevent soil from becoming unnecessarily wet or dry. In those circumstances he suggested that it was preferable to select the plant most suitable for the soil and its moisture content in practice, rather than to adjust the moisture content to suit the plant or the design considerations. In that way road engineers would at least ensure that they had good compaction, which it was generally agreed was essential in embankment construction.

On p. 227 the Authors had suggested that clay was the least suitable material to use in an embankment. That suggestion was based on two reasons: the difficulty of controlling the moisture content of clays, and their unsuitability in high embankments. For the reasons which he had already given, he was not satisfied that those were satisfactory premises on which to base that opinion on clay soils. He had found that clayey sands, clayey gravels, and even sandy gravels could be just as difficult as clays, since they were much more sensitive to small changes in moisture content and easily developed pore-water pressures which made them very unstable.

He strongly agreed with the Authors' statement that embankments should be built to their ultimate dimensions. The widening of narrow embankments could be a difficult and costly operation. In dealing with peat on p. 230 the Authors had not referred to secondary consolidation

effects, which were so important with that type of material. With regard to the construction of roads on natural slopes subject to landslips, he had found that, in general, satisfactory remedial measures were impossible to carry out in practice for economic reasons.

Dr H. Q. Golder emphasized the statement that a bank could be designed with a certain amount of confidence. In some cases the confidence could be greater than in others. Mr Maclean had already referred to the difficulties in predicting the conditions and the strength which would be obtained in a bank some months or years after its construction, and Dr Golder thought that the only way to attempt to measure the strength of the soil was to build a trial bank. He realized the difficulties in that suggestion and that it would be expensive, but such a bank would not need to be very high.

Inside a bank there would probably be a core of material which did not soften, and it was probably the softening effect which was going to be most serious. He suggested a clay bank built with fairly heavy equipment, one side sloping up in such a way that a loaded scraper could go up it, and the other rather steeper, which the empty scraper would come down. At intervals of time one could take samples from that bank at different depths and measure their strengths. That was mentioned in the Paper, and on reading through the Paper he had the impression that it had been done, but to the best of his knowledge it had not. If it were done he thought that a higher bank could be designed with more confidence.

If a slip occurred the strength of the material could be determined, although in most cases it was then too late. But sometimes the bank had to be reinstated and it was then possible to design with a comparatively low factor of safety. He would quote one case to prove that the ordinary method of stability analysis was reliable. He had once inspected a bank which had slipped. It surrounded a building in an ordnance factory and was about 25 feet high, with about a 1-in-2 slope, and was made of clay. The calculated and measured strengths had been in reasonable agreement. It had not been possible to move the toe of the bank out more than 12 feet, because there had been an excavation and a retaining wall below that The authorities had been persuaded that the width at the top could be reduced from 8 feet to 5. By putting a berm in the bank the factor of safety could be raised to 1·2. That was the most that could be done and he had suggested that the bank should be rebuilt to that section. Three days later, however, the engineer concerned had telephoned Dr Golder and had complained bitterly that his bank had fallen down again. Dr Golder had been worried because he had thought that the methods used were sufficiently reliable, and so on the telephone he had taken the engineer step by step through what had been done, finally asking " You rebuilt the bank to the profile that we suggested ? " " No," was the reply, " we built it to the original profile, and we were going to cut it to your profile afterwards." They had got it to the same height as before, and at the

same height it had fallen down again. That was what Dr Golder meant by a reasonable degree of confidence in design.

Mr A. H. Toms observed that since the aggregate of earthworks on railways probably exceeded those connected with any other form of transport, he would like to offer a few remarks on his experience as a railway engineer.

The problems on the Southern Region, of which alone he was competent to speak, were generally those which arose from excavation through, or the use of, stiff fissured clays in which softening occurred because of the release of original pre-consolidation pressure and the absorption of water migrating in the fissures. The origin of these fissures was somewhat obscure but they might be attributable, in part at least, to the ancient Wealden strains.

All types of drainage systems had been resorted to from time to time, but experience had shown that in cuttings the buttress type of drain, namely stonefill trenches, which could but collect such water as flowed into them, might do little to prevent the clay in between them from softening and slipping down between the drains. Anyone who travelled on the Morden South–St Helier line could see that for himself, because various types of drain had been employed there, and it was only mechanical support by arch-type concrete structures, rather like a viaduct lying on its side, which had held the top of that London Clay cutting, which had originally been taken out to 2 : 1 slopes. It might be said in defence of those drainage trenches, however, that they did not minimize the build-up of hydrostatic pressure in the soil fissures, which he considered very important. At Haywards Heath, in the cutting on the up side, north of the tunnel, Mr Toms had been able to put a 12-foot stick down a tension crack almost vertically in the stiff fissured clay, which was full nearly to the top with water. The corresponding hydrostatic pressure was quite high in relation to the mass resisting the movement.

Cantilever sheet-piling, although not a very strong form of construction in itself, unless very stout, was even so, he felt, a more logical form of stabilization for slips, where good resistance to forward movement could be obtained, than a retaining wall which, though it might look massive, weighed little more than the equivalent soil displaced. A retaining wall must be very well founded to resist forward displacement and upheaval of the soil in front of it. There had been good examples of that on railways. One was at Arundel where the wall on the down side had been torn through in two places and the track upheaved in front. Pending the driving of steel sheet-piling as a permanent stabilizing measure, along the original line of the back of the wall, a large mass of old rails have been deposited temporarily on the track being upheaved and the wall had been strutted off them. His personal opinion was that the safest course with an unstable cutting was to trim back the top of the slope, if the adjoining land could be obtained at a reasonable price. The slope could

be formed to a berm shape, which was most economical from the point of view of excavation, so long as adequate drainage was provided, both behind the top and on the berm, to minimize the seepage of water into it.

He would like to refer to the glauconitic herring which had been dragged across his path by Mr Glossop. Mr Toms's copy of the Paper by Drouhin, Gautier, and Dervieux, to which the Authors had referred, had been mislaid for 3 years and had come back into his possession only the previous day; he had re-read it at once. He could not say whether the zeolite action of glauconite had contributed in any way to the Folkestone Warren slides. Since writing the Paper [1] which had been presented to the Institution a few years ago, a great deal more research had been done at the Warren by very deep borings, and another Paper had been written by him which he hoped might find its way into the Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering. That research showed that the predictions which had been made as a result of the original work had been vindicated. There was very high hydrostatic pressure at the back of the slides from the water which came from the chalk above, tending to cause slipping, which far exceeded the pressure of the greensand water below, and toe erosion of the foreshore was an aggravating cause.

Irrespective, therefore, of such glauconitic effect it seemed logical to pursue the courses of action adopted by the Railway Executive as a result of that research, which were :—

(*a*) To provide erosion protection and additional weight on the foreshore in front of the old sea wall in the form of a new sea wall some distance seawards of the old one and compacted filling between them protected with concrete.

(*b*) To explore the practicability of lowering the water table in the back of the slips by appropriate extension of the existing headings.

The Authors had referred to a slip slumping down and attaining a state of stability. Mr Toms emphasized that if there was a progressive build-up of water in the back of the material after slipping it never attained a state of real stability, unless one did something positive about it, such as drainage, toe weighting, a retaining structure, or re-forming to an improved profile to reduce the disturbing forces.

Seasonal swelling seemed to be the principal cause of slow-creep movements of clay banks and many retaining structures, which proceeded for years before final rapid failure occurred.

Slips in cuttings through stiff fissured clays were frequently deep, several important ones on the Southern Region having upheaved the floors of the cuttings. Close-packed brickbats might be a satisfactory

[1] A. H. Toms, "Folkestone Warren Landslips: Researches carried out in 1939 by the Southern Railway Company." Railway Paper No. 19, Instn Civ. Engrs, 1946.

facing to the sides of " buttress " drains in clay slopes, but hardcore as a backfill for line-side drains was quite unsatisfactory, becoming choked with clay in a short time. For track drains a wall-block and channel-drain construction, with minus-$\frac{1}{4}$-inch rock-crusher residue between it and the clay, was behaving well after periods of up to 5 years.

Chalk had proved to be an unsatisfactory filling for drain trenches or track formations, on account of solution and frost action.

Trees on slopes were open to the following objections :—

(*a*) Evaporative effect was least in winter when needed most.
(*b*) The roots of even large trees had frequently proved insufficient to arrest deep slipping.

Rail traffic was often kept running over slipping embankments by continually tipping new material on top. Though expedient, that procedure contravened soil mechanics principles, since no positive factor of safety resulted and water-retaining pockets of pervious materials were formed on top of the banks.

On the Southern Region, toe berms were now constructed wherever possible, to give positive support. The immediate difficulties were usually : (*a*) lack of road access to the toe of the slip, or (*b*) the cost of labour required to manhandle the filling down the banks from trucks on top—no satisfactory mechanical aid having yet been evolved for the small isolated jobs. Berm construction along weak banks on the Southern Region had paid ample dividends in elimination of winter speed restrictions.

Considerable benefits could be derived from drainage of the water pockets formed in embankments. They should, however, be explored beforehand by vertical borings.

In any new construction, waterproofing of the tops of embankments might prove to have long-term economic justification.

Mr Toms thought that the word " dilatant " as applied to sands by the Authors on pp. 240 and 243 was a misnomer, since the sands referred to had been compacted when disturbed. That was the reverse of dilating.

The statement on p. 240 that water modified the angle of friction of quartz grains was incorrect. The angle of repose of quartz grains in water was virtually the same as in air. Mr Toms had proved that by tests which he had made 20 years or more previously.

Mr E. Eric Morgan, commenting on the reference made by the Authors to the use of berms for slope correction, gave an example of a practical application of this method by describing works in hand on a slip at one of the sites referred to by the Authors in their acknowledgements.

The slip had occurred on the Dorking by-pass road, the footpath of which had subsided over a length of nearly 40 yards. When the ground had started to move, one-half of the 30-foot carriageway had been roped

off, but a few weeks later, after movement had ceased, the width of the traffic lanes had been increased to 22 feet. The slabs had lost some support near the kerb and pressure grouting would be necessary after the bank had been reinstated.

The road at that point was carried by a sand embankment about 30 feet high across the top end of a narrow valley, which fell from west to east. Before the embankment had been built a surface-water drain had been laid from the upper to the lower side. The soil forming the bed of the valley was a semi-permeable sandy clay, overlying heavier clay. In late winter the ground-water was near the surface. It percolated through the soil under the embankment to a pond on the opposite side, situated close to the toe of the bank. From the overflow of the pond there was a piped connexion to an outfall lower down the valley. Because of the pond, the water table at the toe of the bank was always near the surface of the ground.

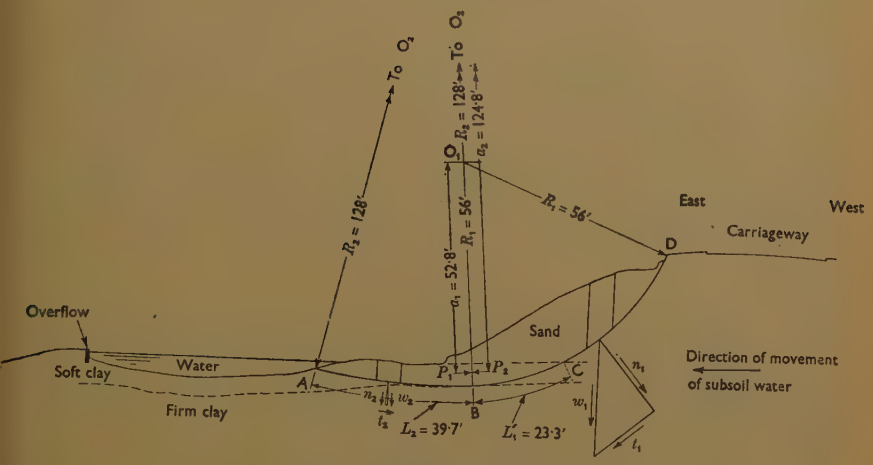
The Dorking by-pass road had been constructed about 20 years ago. Since then there had been a gradual softening of the clay until it became too weak to support the weight of the bank above, and eventually, following a wet period, the soil slipped. It was instructive to note that the direction of the slide was not normal to the line of the bank but radial towards the centre of the pond, the bed of which rose considerably.

The method adopted for the repair of the embankment had been to flatten the slope by the inclusion of berms. The work was nearly completed. The pond had been drained, and all soft material removed and replaced with a layer of hardcore with open-jointed drains connected at a low level to the old overflow pipe. That had ensured that the filling to the pond would remain in a drained condition. The Authors had mentioned the need for caution when excavating close to the bottom of a slipped bank. Along the toe of the slipped bank a graded filter drain had been constructed to collect subsoil water coming from the bottom of the bank. The outlet from that drain was connected directly to the old overflow drain with close-jointed pipes on concrete, to avoid discharging into the area previously occupied by the pond.

The bank was now being reconstructed with a berm at about one-third of its height. The filling to the pond had also been raised to form a berm, thus helping to make the bank stable over a gradually sloping surface consisting of clay, except over the hardcore layer in the bed of the pond. Tests on the softened clay taken shortly after the slip occurred produced a shear value of about 250 lb. per square foot. The sand in the embankment had an angle of shear resistance of 35 degrees.

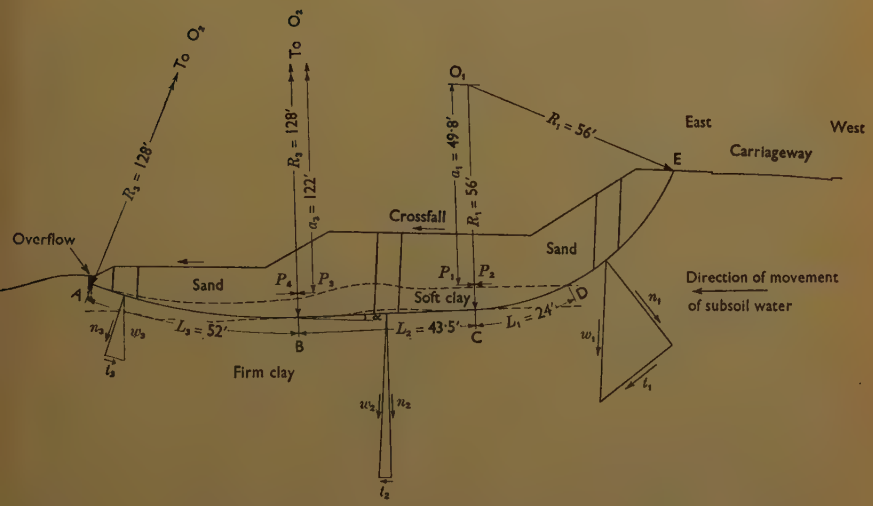
The probable slip plane, as found by borings, formed two circular arcs of different radii (see *Fig. 14* (*a*)). The calculated factor of safety for the profile after the slip was 1·1, thus indicating that the strengths obtained from the samples were probably representative of the whole. The formulae used for calculating the factor of safety and the following stability

Fig. 14 (a)



SECTION THROUGH SLIPPED BANK

Fig. 14 (b)



BANK AS NOW RECONSTRUCTED

analysis were derived from two given by Cooling and Golder [1] in 1942. For the new profile the probable plane of least resistance would be represented by two opposing circular arcs of different radii connected by a sloping line (see *Fig. 14 (b)*). Calculations indicated a factor of safety of 1·4 without taking into account the beneficial effect of the hardcore in the pond.

Much of the preliminary work, including the extremely laborious stability calculations, had been undertaken by Mr J. C. Waddington, B.Sc., A.M.I.C.E. The constructional work was being carried out by direct labour under the direction of Mr A. B. Saunders-Jacobs, B.Sc.(Eng.), A.M.I.C.E., within whose area the slip was situated. Mr Morgan wished to thank the Authors, whose advice on the proposed works had been most helpful.

Mr F. L. Cassel observed that the Paper was very comprehensive, considering its wide scale and the narrow frame into which it had had to be compressed. As Mr Toms had already indicated, almost all the problems dealt with in the Paper played the same role in railway engineering, and some even more so, because, owing to the narrower limitations with regard to gradient and alignment, cuttings and banks were more numerous, and were often longer and deeper than in road engineering. Mr Cassel mentioned that because his own experience referred largely to railway engineering.

With regard to cuttings in clay, the chief remedial measure for slips in such cuttings—stone-filled counterforts—had already been mentioned. Their usefulness had been recognized for a very long time. They had, of course, as the Authors had said, to be carried to a depth sufficient to intersect the failure plane. They were often regarded as a drainage measure. Mr Cassel's personal view, which he thought Mr Toms had confirmed, was that that was only a secondary effect in so far as they helped to carry surface water away rapidly and perhaps also water entering anew from the top, if it was led to them by herringbone drains or other devices. For the drainage of the slip plane as such, however, they were usually quite ineffective, except where water moved in fissures. A clay which had softened to a plastic state was almost undrainable, and the effect of a drainage channel would extend for only a few inches sideways.

He believed that the counterforts served their purpose because they cut a large slipping mass into a number of short slipping bodies. Those had to overcome, in order to continue their movement, the shearing resistance not only in the slip plane but also laterally along the counterforts, which were rigid and not slipping. The resisting area would be greatly enlarged, and, if the counterforts were near enough to each other, it might even be doubled. It was a well-known fact that a minimum distance between the counterforts was required to make them effective, and for that distance figures of between 20 feet and 40 feet had been

[1] L. F. Cooling and H. Q. Golder, "The Analysis of the Failure of an Earth Dam during Construction." J. Instn Civ. Engrs, vol. 19, p. 38. (Nov. 1942).

given—and had proved suitable, depending on the local conditions. Did the Authors take the same view about the usefulness of counterforts ?

They had mentioned the special case of slips in stiff fissured clays and the theory proposed by Dr Skempton about the development in time of such slips. Mr Cassel very much admired that brilliant mental exercise. According to that theory, the deterioration of the clay was governed by the depth of the cuttings and the time passed since their construction. He had argued against that theory at the time when it had first been put forward. He did not think that his argument had made much impression, but he was going to continue it, lest anybody should believe that one could estimate, from a knowledge of age and depth, the time at which a slip in a cutting in stiff fissured clay might occur, even within very wide limits.

There were literally thousands of such cuttings in Great Britain, and only a relatively small number had failed or were failing. He knew of a number of very long cuttings of that type—some of them half a mile and more in length—where perhaps 5 to 10 per cent of the length might have failed at some time—some a long time ago, others recently. The remaining 90 per cent were of the same depth and age, and were not failing to any extent. That seemed to prove that, apart from age and depth and, of course, the properties of the particular clay, some other agent had a predominant influence, which was probably purely local and topographical, probably the ground-water movement, previous and present, at the particular locality. Would the Authors express a view on that opinion ?

Had the Nixon Extruder, shown in *Fig. 2*, been described elsewhere ? Was it used to extrude the soil core more from the Proctor cylinder against the coring tubes, or were these pressed into the cylinder ? It was not possible to see that from the photograph in the Paper. How many testing specimens were obtained at the same time ? He believed extruding the soil core against static coring tubes to be preferable, and he had used a rather primitive device of that kind successfully, believing that it produced less disturbed testing specimens.

*** **Dr A. W. Skempton** observed that the Authors had made a very useful survey of the various problems of stability as encountered in cuttings, embankments, and natural slopes. It was necessary that, from time to time, someone of wide experience should pause to consider the whole field of a group of related problems and the Authors had carried out that task in a comprehensive manner.

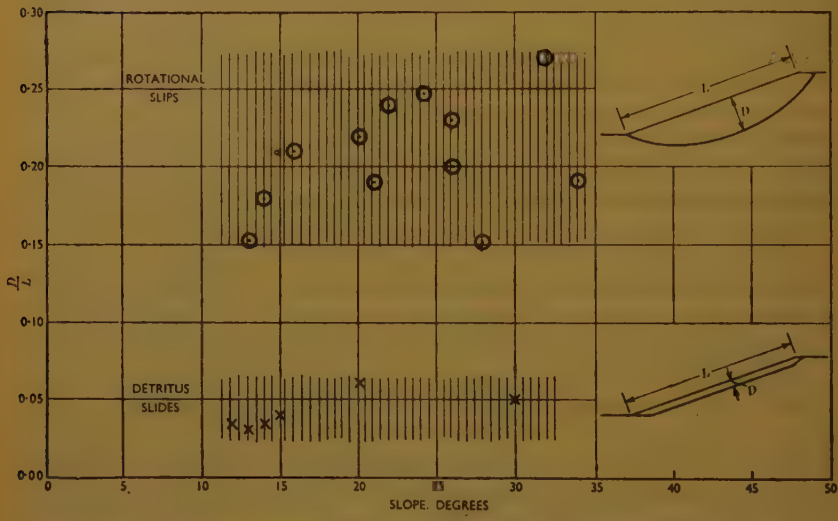
Dr Skempton confined his remarks to two questions concerning the stability of natural clay slopes. During the past 15 years he had had the opportunity of studying a number of landslips in clays and he had been impressed by what seemed to be a remarkably clean-cut division of such landslips into two major types. The first type was that of the well-known

*** This contribution was submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

rotational slip, where failure occurred by shear along a relatively deep curved surface. A great deal had been written about such slips and Dr Skempton wished to add to the available information only the fact that, in the examples known to him, the ratio of the depth of the slip D to the length L measured up the slope lay between the limits 0·15 and 0·28. The ratio was independent of the inclination of the slope and of its height (see *Fig. 15*).

The second type was that of the flow slide or detritus slide, where failure occurred by shear along a relatively shallow plane surface parallel with the

Fig. 15



Geometrical Proportions of Rotational Slips and Detritus Slides in Clay

slope. That type of landslip had received less attention than the rotational slip but was probably of equally frequent occurrence. It consisted essentially of the downhill movement of a sheet of clay, and in marked contrast with the proportions of the rotational slip, the ratio of D to L in the detritus slides lay between the limits 0·03 and 0·06 (see *Fig. 15*).

It would be seen that, in their proportions, the two types of landslip were completely distinct; and there could be no doubt that two entirely different mechanisms were involved. The practical consequence was that the remedial measures were also different in the two types of movement. The most effective measure for stabilizing a rotational slip was to reduce the disturbing moment, either by removing earth from the top or adding weight at the toe, or both. But with the sheet-like movement of the detritus slides, that procedure would hardly improve stability, and the only effective

measure was deep drainage by means of trenches taken down to the undisturbed ground and back-filled with granular material. For small-scale rotational slips that method could also be used (as in railway cuttings), but it was impracticable for large rotational slips where the value of D might be 50 feet or more.

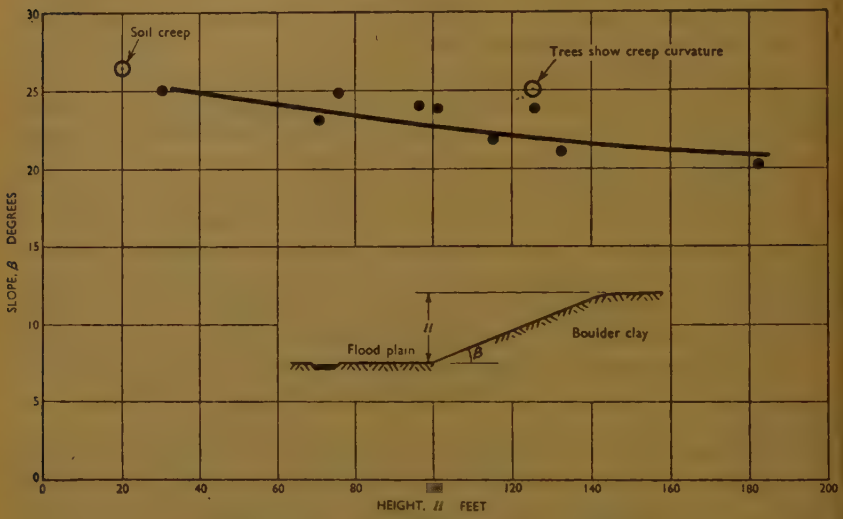
The second question was that of the final stability of clay slopes. Analyses based on undrained tests (of which the $\phi = 0$ analysis was an important particular case) gave the factor of safety of a clay slope immediately after construction, when little (if any) dissipation of pore pressure had occurred. In the course of time, the pore pressures would decrease and that process might lead to either a decrease or an increase in stability. In embankments, the factor of safety usually increased with time, but in cuttings the opposite was more often true. Consequently the conditions obtaining many years after excavation had to be investigated. In stiff fissured clays that problem was difficult, but of great importance because such clays were often encountered and also because the drop in strength with time was known, in some clays, to exceed 70 per cent of the original value when the cutting was first made. Field evidence was being obtained, however, which, it was hoped, would lead to a practical solution of the problem.

The case of non-fissured stiff clays was also important, although little work had so far been done on the final stability of those clays. The boulder clays of northern England and Scotland were good examples of such materials. In principle the final stability could be analysed in terms of drained shear test results and a knowledge of ground-water conditions. But few analyses of that type had been presented in the literature and when, a few years ago, Dr Skempton had been faced with the problem of deciding upon the final stable slope of the sides of a deep valley in boulder clay he had had resort to field observations in the immediate neighbourhood. At the particular site under investigation, where it had been proposed to build a road bridge over the valley, the slopes were obviously unstable, and the stream was still actively eroding the toes of the slopes. Yet within a few miles of the site, in the same clay, a flood plain had been formed, and at several locations the slopes had been free from stream erosion for a long period—as proved by the age of the trees growing on the plain. Therefore those slopes had attained what might be called permanent stability, on a human time-scale even if, on a geological time-scale, they would attain still flatter inclinations.

The stable slopes had been surveyed, and the results were shown in *Fig. 16*. In two cases soil creep was active, but in the other nine cases there was no evidence of creep. The stable slope had decreased slightly with increasing height, as might be expected, and by interpolation the safe angle for the slopes at the site of the bridge had therefore been obtained. In that way, by making use of simple field observations, reliable information had been gained without the need for any laboratory testing. Yet,

in general, it was most desirable to have methods of predicting final stability from tests on samples, and there was considerable scope for further research in that aspect of the problem of clay slopes.

Fig. 16



Stable Slopes in Boulder Clay. Durham

The Authors, in reply, said that Mr Bradbeer's comments on the construction of embankments using clay and fine sand were very valuable, since they represented an enlightened point of view based upon a wide experience. There was no doubt that, especially when working with difficult soils, there were great advantages in billing the work of compaction at daywork rates, for that allowed the engineer to retain complete control of the work and, in the long run, resulted in economy, since the contractor no longer felt it necessary to allow a wide margin for contingencies when making up his price. It also enabled the engineer to suit the progress to weather conditions, and that in itself was a matter of necessity if good results were to be obtained. The success of such measures in practice was well illustrated by the cases quoted by Mr Bradbeer of high embankments of Weald clay and mixtures of Tunbridge Wells and Ashdown sands.

Soils of the silty-sand type, such as those described by Mr Bradbeer as occurring in the Ashdown beds, were exceptionally difficult to deal with in practice. During excavation perhaps the best method was to programme the work so as to allow the excavation to proceed in stages, with intervening periods for drainage to occur. Where outcrops of such material were the cause of instability of slopes, graded filters might be useful.

The Authors agreed with Mr Maclean that when the programme for

much-needed major roads was started, soil mechanics would certainly play a very important part, both in design and in construction. For example, it seemed likely that stabilized soil sub-bases would be used on weak sub-grades. They agreed that some sections of the Paper had necessarily been not more than brief references to their subject, but it had been prepared along the lines suggested by the Divisional Board.

They considered that the most difficult design problems in respect of both slope stability and settlement were associated with high embankments, which they agreed might be defined as those above 30 feet. On the other hand, widespread trouble had been observed on low banks, and during the preparation of the Paper a concrete road had been examined which had cracked badly and had very poor riding qualities, although carried on a bank only 3 to 4 feet high.

The selection of the moisture content at which filling should be placed was a matter calling for the most careful consideration, especially where material containing a fine-grained fraction was to be used in a high bank. The suggestion that the best moisture content at which filling should be placed was a little less than the optimum value, was based upon a compromise between high initial shear strength, minimum softening, minimum primary settlement, least effect of saturation in producing further secondary settlement, and avoidance of pore-water pressures during construction and subsequent consolidation. It was not intended to give the impression that all those desiderata were related to a single moisture content or that the maximum shear strength would be obtained. Maximum strength was usually obtained at moisture contents lower than those to which the other properties were related, and it appeared most practicable to adopt a placement moisture content which satisfied the other requirements and to accept a somewhat reduced strength. Whilst *Figs 1* and *4* (used in the Paper to illustrate the variations of strength with density and moisture content) were based upon tests in which the compactive effort was equivalent to that used in the British Standard compaction test, it was believed that similar relationships would hold for other compactive efforts. Where mentioned elsewhere, optimum moisture content was that appropriate to the method of field compaction to be used in actual construction. The first step in the Holtz method for evaluating placement moisture content was to devise a laboratory technique which would reproduce the moisture/density curve developed under field conditions.

That was only one approach to the problem, and Mr Maclean had suggested an alternative in which the filling was compacted at the moisture content it was estimated it would ultimately reach. That, however, still posed the question as to what the ultimate value would be. One method of evaluating such conditions would be the application of the moisture/suction principles developed at the Road Research Laboratory, but whilst the correlation between calculated and measured moisture distribution in fine-grained soils covered with an impermeable surfacing had been

demonstrated by laboratory tests to be excellent, much work remained to be done before the theories could be employed with confidence to estimate accurately the moisture conditions within an earth structure with variable and possibly largely indeterminate boundary conditions.

It was queried whether the idea of compacting soils at their natural moisture contents was generally accepted, as had been suggested by Mr Maclean. It was the Authors' opinion that fundamentally that was not the correct procedure, although in some instances it would yield good results. " Good compaction " presumably signified low air voids, but that did not necessarily involve high shear strength—surely a prerequisite of good-quality filling. Further, even a fill material, uniform in type, would exhibit variations in moisture content, so that if good " compaction " was obtained by varying the compactive effort—it was admitted that that did not demand a wide range of plant but could be achieved by altering the numbers of passes of a few machines—an earth structure would be produced in which the shear strength and also the compressibility were non-uniform. Surely it was worth the trouble of controlling the moisture content and obtaining optimum performance from plant to place a fill with uniform properties ? Under certain conditions, no choice of plant might be possible, and the engineer would be forced to control the materials in order to secure satisfactory compaction.

It was agreed that placing difficulties were met not only with clay, but also with predominantly granular soils. Pore-pressures did develop in such materials which could become unstable under a roller, but the effects were of less consequence than with clay soils, and of a shorter duration, so that interference with progress was likely to be less.

The Authors agreed that remedial works in landslip areas were likely to be impracticable and uneconomical. Such areas were best by-passed.

They would assure Dr Golder that they had not in fact built a trial bank without informing him of the fact. The Paper was intended to suggest that in the present state of our knowledge that was the best approach to a difficult problem, and they were relieved to hear that he was in agreement with them. They would emphasize that ample time should be allowed for changes to occur in the condition of the fill so that the best possible use was made of the trial bank and no misleading premature conclusions were drawn.

The Authors agreed with both Mr Toms and Mr Cassel that the main function of the counterfort drain was to increase the resistance to movement by dividing the slipping mass into a number of sections and introducing friction planes between the vertical sides of the granular drain filling and the adjacent soil masses. The degree to which they were effective as drains in reducing pore-water pressures depended entirely upon the nature of the soil and might, in fact, be negligible.

The reference in the introductory remarks to the effect of glauconite on the Folkestone slips through the medium of the ground-water had been

designed to provoke discussion, since there was no actual evidence that any such effect existed there. The Authors welcomed Mr Toms' account of recent research at Folkestone Warren, and no doubt the physical causes to which he attributed the movement were indeed the most important. There remained, however, the possibility that the chemical composition of the ground-water might play a minor role in producing instability, although its effect might be negligible compared with those of the high hydrostatic pressures at the back of the slips, and of the pore-water pressures in the underlying Greensand.

There had evidently been a misunderstanding in the interpretation of the statement on p. 240 that water altered the angle of friction of coarse grains. The Authors had referred to static friction between adjacent quartz surfaces and it had been proved by experiments, described by Tschebotarioff at the Rotterdam Conference in 1948, that the coefficient of static friction was higher for the moist and saturated materials than for the dry condition. Mr Toms was presumably thinking of quartz grains in bulk, tested using normal soil-mechanics techniques which evaluated the angle of shearing resistance. That parameter was influenced by the degree of interlocking of the grains as well as by static friction. Generally speaking, it was agreed that the angle of shearing resistance was not significantly affected by moisture content, but recent work on the subject had yielded somewhat conflicting data.

Mr Morgan's analysis of a slip on the Dorking by-pass road had brought out several interesting points. The failure had been caused, in the Authors' opinion, by the softening of the underlying clay, and by an increase in the bulk density of the fill during a wet winter. It was interesting to note that a period of 20 years had elapsed after construction before the combination of circumstances led to failure. With regard to remedial measures, the use of a filter drain to intercept seepage and a berm to stabilize the slope, which could not be flattened, were effective measures. The comparatively large width of the berm was a feature to be noted, and was necessary because of the very low shear strength of the clay stratum beneath the embankment.

Mr Cassel had commented on the similarity of the type of soil-mechanics problems found in road and railway engineering. Broadly, that was true, but the consequences of settlement, for example, were very different. Settlements of a railway embankment amounting to several inches were easily corrected in the ordinary course of maintenance, but similar movements of a bank supporting the concrete slabs or even the flexible pavement of a modern road was a serious matter, and could involve quite extensive repairs. The Authors agreed with Mr Cassel that the point at which a slip occurred in a long cutting would almost certainly be determined by some local peculiarity of the site, but they believed that the broad principles enunciated by Dr Skempton were useful in considering such problems.

The Nixon Extruder had not yet been described in detail, although it

had been in use in the Authors' laboratory for several years. In principle, the soil was extruded by a hydraulic jack from a sampler tube or from a Proctor cylinder against static coring tubes. As many as four specimens could be cut in one operation. It had given very satisfactory results.

The Authors appreciated Dr Skempton's accounts of the classification of landslips in clays and the estimation of the final angles at which clay slopes would remain stable.

Failures of natural clay slopes with which the Authors' firm had been concerned had not, unfortunately, been such as to permit accurate evaluation of the D/L ratios, In four cases, however, a liberal interpretation of the records had indicated that the range of approximate values for deep-seated slips was from 0·19 to 0·35. On the one occasion when a flow slide had been examined, the D/L ratio was between 0·05 and 0·075. The remedial measures advocated by Dr Skempton were in line with those described in the Paper.

The difficulties of forecasting quantitatively the changes that were likely to occur in clays with time had been mentioned by several speakers but the necessity for reliable estimates was stressed by the statement that in some cuttings in fissured clays the soil had suffered a drastic reduction in strength during the life of the cutting to something less than 30 per cent of its original value. Further, it was inferred that even with non-fissured clays, satisfactory analytical methods were not available in the present state of knowledge. The development of reliable methods based upon the measurement of soil properties was certainly a matter that should be diligently pursued.

Dr Skempton was to be congratulated on the manner in which he had approached the problem of determining the safe slopes in boulder clay in Durham. His remarks constituted a noteworthy reminder that soil mechanics remained essentially a practical science and that much valuable data was still to be obtained from careful observation of conditions in the field. That was emphasized by the reference to the collection of site data in relation on the behaviour of fissured clays.

Correspondence

Mr P. E. Fairbairn observed that in their reference on p. 243 to a case * of internal erosion in clay (ascribed to operation of base-exchange processes in an overlying glauconitic bed), the Authors had quoted Proix-Noé as having stated that some of the calcium in solution was replaced by potassium on coming into contact with the glauconite. That quotation was in agreement with the description of the landslides by Drouhin, Gautier, and Dervieux, but it should be pointed out that the earlier Paper by Proix-Noé contained no specific reference to potassium as the operative base. To quote Proix-Noé : " on coming into contact with glauconite fixation of

* See list of references p. 251

calcium ions takes place with liberation of alkaline ions. . . . the alkaline waters act as a dispersing agent on the colloidal (clay) micellae and bring about hydrolysis of the aluminium silicates."

Without entering into a discussion on the probable effectiveness of the base-exchange process in glauconite as a cause of landslides, it seemed reasonable to suggest that sodium rather than potassium ions might have been the effective agent. Although glauconite was essentially a potassium mineral containing up to 8 per cent of K_2O, it had been found experimentally that not more than 10 per cent of that K_2O content was available for base exchange, whereas Na_2O could replace K_2O in some glauconites up to a total proportion of 3 per cent. A convenient summary of the properties of glauconite was contained in Wartime Pamphlet No. 33 of the Geological Survey, " Glauconite Sand of Bracklesham Beds, London Basin," by K. P. Oakley, May 1943 ; in discussing the commercial use of glauconite as a water-softening agent that pamphlet pointed out that normal or potassic glauconite had little or no water-softening capacity, and required " activation," or treatment, with a strong solution of sodium chloride to effect substitution of replaceable potassium ions by sodium ions. Mr Fairbairn noted that in Proix-Noé's analysis of the water circulating in the marls the quantity of sodium present was more than three times that of potassium (0·183 gramme as against 0·051 gramme per litre).

Proix-Noé's picture of the second stage of the process, already quoted, scarcely deserved the description of base exchange given to it by the Authors. Although base exchange was not necessarily inoperative at that stage, Proix-Noé did not mention it, and clearly envisaged the process as one of simple peptization of the marl by the action of an electrolyte, coupled with breakdown of the clay molecules to give free SiO_2. That was hardly base exchange, in the normal accepted use of the phrase.

Mr C. A. U. Craven said that a working theory for the origin of some of the landslips in the Millstone Grit area of the Pennines could be developed from the principles set out in the Paper, together with a knowledge of the local geology.

That tentative theory was drawn from experience of a limited area around Chapel-en-le-Frith and Castleton in Derbyshire, the scene of many notable landslips. Except for the slip east of Mam Tor, the major landslips in that area were concentrated on the north-west flank of the ridge extending north-eastward from Mam Tor and the north-west flank of Combs Moss, the high ground north-west of Buxton. The north-westerly attitude of those slopes caused them to receive less solar radiation than the opposing ones, so that during the final waning stage of the last glaciation, ice would survive longest on them. Therefore those slopes would be left in a more unstable condition than those over which melt-water had flowed and solifluxion occurred which had redistributed the loose detritus and shattered surface rock into a more stable form. The landslip east of Mam Tor, although on the south-east side of the ridge, was shaded from the sun by Treak Cliff.

The low ground on which Chapel-en-le-Frith was situated was covered by boulder clay, which probably indicated that that hollow was once the site of a glacial lake held in place by an ice-dam blocking the valley which drained it. The Edale Valley and Castleton Valley might similarly have been the site of another glacial lake, although the evidence for that was not so abundant at present as in the previous case.

One could picture, in the final stages of the last glaciation, a landscape in which the southerly-facing slopes had been cleared of ice while the northerly ones were still covered, and in part supported, by slowly melting ice. Meanwhile the low ground at the toe of the last-surviving ice would be covered by lake water. An example of an ice-dammed lake existing at the present time was an arm of the Lago Argentino in Patagonia (Nichols and Miller, 1952).[1] The Paper described how a tongue of ice from the Moreno Glacier flowed across the mouth of an arm of the lake and then periodically collapsed, followed by a rapid drop in the water level on the upstream side.

In the area under discussion, it could be visualized that there would be a very rapid draw-down of the lake level when the ice-dams suddenly collapsed. Such an event could be sufficient to cause a failure of the less stable slopes in the manner of the collapse of an earth dam built with too low a safety factor or inadequate drainage to withstand rapid draw-down.

Mr B. W. Bryan said that base exchange occurred under London and under large areas of Essex where water passed from the chalk formation into the Thanet sand, the calcium bicarbonate being replaced by sodium carbonate, in some cases wholly and in others partially. To allow the exchange it was necessary that salt should be present in the sand, and the water affected was thus deep seated.

Where the sands outcropped, the exchange would not occur owing to any salt present in the past having been washed out.

The action had little, if any, relevance to the matters discussed in the Paper, but full particulars could be found in Memoirs of the Geological Survey, " The Water Supply of Essex from Underground Sources," by W. Whitaker and J. C. Thresh (1916), p. 20.

Mr D. A. Fox, referring to the Author's descriptions of various systems of vertical drainage for accelerating consolidation underneath banks founded on marshy deposits, noted that the advantage claimed for the method of accelerated consolidation, over the method of displacement, was that the saving in fill might more than pay for the installation of the drains.

He suggested that such a comparison might not always work out so favourably in practice, and that the cost of vertical drainage would depend very greatly upon the purpose for which it was desired.

There appeared to be two main uses for vertical drainage :—

[1] R. L. Nichols and M. M. Miller, " The Moreno Glacier, Lago Argentino, Patagonia," *Journal of Glaciology*, vol. 2, No. 11, 1952.

(1) To accelerate consolidation of the weakest upper strata of the subsoil, in order to strengthen them sufficiently quickly to carry the desired load.

(2) To accelerate consolidation of all compressible subsoil in order to reduce or eliminate disruptive settlement effects before foundations proper were constructed.

Vertical drainage could be designed to cover both the above cases, but, especially where the depth of compressible subsoil was great, the costs were also likely to be high. For economy, therefore, it was desirable to decide upon the main function of vertical drainage, and to design it accordingly.

Where, for instance, overloading was likely to be appreciable, case (1) would be the main criterion. The cost of installing vertical drains, and the fill required, would be compared with the fill required for solutions such as :

(*a*) the method of displacement,

(*b*) the provision of counterweighting side berms, or

(*c*) part excavation and infill in lieu of the method of displacement (see below, reference to *Figs 7*), and in those circumstances vertical drainage might be more economical.

Where overloading might be small, and where displacement or plastic settlement might be restrained, or small compared with the consolidation settlement of very thick layers of compressible subsoil, then the fill required with vertical drainage might well exceed that required without drainage. That was because the drains tended to concentrate the active pressure within the drained foundation area instead of permitting dispersal of active pressure with depth as with the " bulb of active pressure."

For example, a 150-foot-diameter steel tank was to be founded on a raised hoggin " pad," underlain with 84 feet of compressible clayey and silty soils. Without vertical drains, consolidation settlements were estimated to range from 3·5 feet at the perimeter to 5·4 feet at the centre. With vertical drains the settlement was estimated to be about 5·9 feet over most of the foundation area. That represented extra fill over and above the cost of installing the drains.

Therefore, where strict economy was essential and the depth of compressible subsoil great, it was necessary to decide whether drainage to the full depth was really essential. Compromise might be feasible in certain circumstances, for example, a few deep drains at wide spacings, interspersed with shallow drains at closer spacings ; the latter would be concentrated in the regions where plastic flow, slip, or displacement settlement were to be expected. Compromising thus for economy in capital expenditure meant, however, some difficulty in determining the possible trend of differential settlements where those had not been " ironed out " during construction. Would the Authors comment on the real necessity for drains more than 100 feet deep under the New Jersey Turnpike ?

The Authors had referred to the cost of special plant required for Swedish wick drains, but Mr Fox believed that the cost of installation compared favourably with the sand drain method used in the United States. Unfortunately, Papers describing vertical drainage seldom included detailed soil data, particularly their composition and Atterberg indices, and coefficients of vertical and horizontal permeability or consolidation. That made economical comparison, and comparison with proposed schemes, rather difficult.

Could the Authors suggest simpler methods of determining horizontal permeability " in situ " for British alluvial and estuarine clays and silts, without the necessity to construct and overload trial banks to discover how a particular vertical drainage system would work ? What spacing of boreholes would be necessary to determine horizontal permeability ? Conversely, did the Authors know of any experiments to determine horizontal permeability in those kinds of soils, by flooding one borehole with dyed water and observing the flow into adjacent drained boreholes ?

With reference to the methods of displacement of peat illustrated in *Figs 7*, in certain circumstances it might be practicable and economical in marsh clays to excavate ditches parallel to a proposed bank, locating them below the centres of estimated slip circles. Infilling with well-compacted granular soil could provide a greater resistance to shear around a slip circle than could be obtained with the natural soil, even when drained and consolidated under the overload filling. Excavation by dragline along the line of the bank and ditch, with infill tipped in a similar manner (*not* from the edges) and following directly on excavation, might permit the attainment of quite appreciable depths in marshy deposits, with fairly steep temporary side slopes.

Mr B. Jakobson, referring to the use of loading berms and the displacement of weak soil by loadings, submitted the following description of certain aspects of Swedish practice.

Loading berms were used irrespective of the actual depth of the weak foundation soil. The design diagrams given in *Géotechnique*[1] referred to foundation soils of constant shear strength, where the necessary width of the berms was proportional, for a certain depth, to the depth to firm underlying ground. However, investigations of shear strength, especially those made in recent years by means of the vane borer, had shown that, as a rule, the shear strength of soil increased appreciably with depth. That allowed a considerable reduction in the necessary width of the loading berms—a factor which would therefore be considered in the design. It had not been possible, however, to prepare design diagrams applicable to that condition.

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[1] B. Jakobson, "The Design of Embankments on Soft Clays." *Géotechnique*, vol. I, No. 2 (Dec. 1948), p. 80. Unfortunately there has been a dislocation along the x-axis in reproducing the diagram for case II (*Fig. 7*, p. 87); the values given for $\frac{b_1}{D}$ along the x-axis 0, 0·5, 1·0, and 1·5, should be 0·1, 0·6, 1·1, and 1·6.

It was borne in mind, first, that loading berms were generally used only when the bearing capacity of the ground, allowing a suitable factor of safety, exceeded at least half of the intended load, and secondly, that loading berms did not prevent, but increased settlements. Thus if no settlements were tolerated, other methods had to be used.

When displacing a weak soil by loading, attempts were made to mini-

Fig. 17

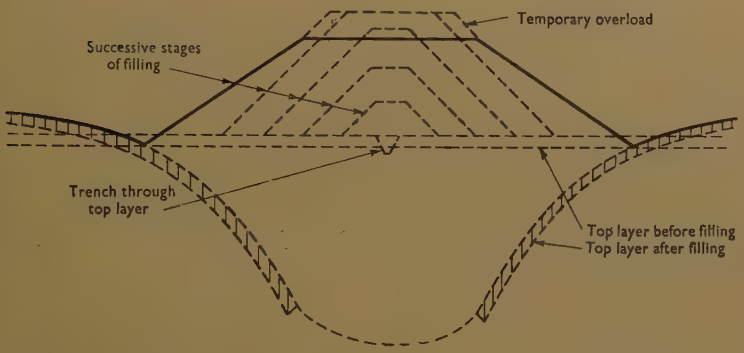
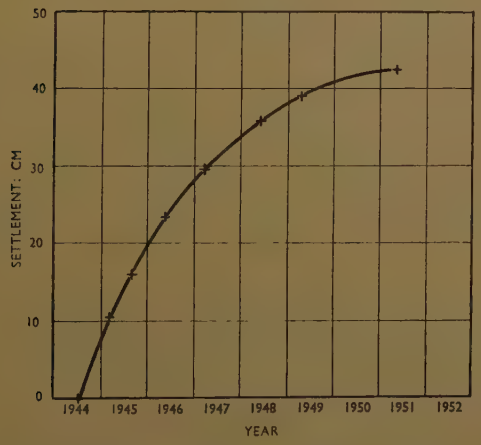


Fig. 18



mize the amount of earth filling, especially where the filling had to be imported from a borrow-pit. Where the top layer of the soil was held firmly together (for example, by plant roots), a narrow trench was usually excavated—much narrower than that shown in *Figs 7*. The fill was then placed symmetrically, starting on the centre-line of the proposed bank, and maintaining the side slopes as steep as possible ; the top layer, which was

pressed downwards, prevented the earth filling from spreading, and resulted in a cross-section as shown in *Fig. 17*. Since a temporary overload, greater than the final load, was always applied to the top of the bank, sufficient stability was obtained, although the filling was sometimes not forced completely down to firm ground. That resulted in a " floating bank," and settlements could not be avoided ; but it had been found very difficult to avoid settlements in any case, even when using " bog-blasting " methods.

Fig. 18 showed the settlement curve determined by accurate levelling of a large bank-fill across Lake Aspen. That bank, mentioned on p. 233 of the Paper, had been forced down through clay by blasting with about 6 tons of dynamite in 1944. As shown in *Fig. 18*, the settlement had reached about 40 cm. since that time, and the rate of settlement was, at the present time, less than 1 cm. per year.

Mr W. H. Ward, referring to the Author's mention of the possible effects of base exchange of natural waters on clays, observed that one of the common situations in which that condition might arise could be in a clay dam, retaining water that was different from the ground-water where the clay had been dug.

That possibility had occurred to him when inspecting a small dam failure at Harrogate (described by Davies [1]). He had noticed the unusual occurrence of heather growing on the dam, whereas it did not grow on the same clay in the adjacent field from whence the dam material was reported to have been dug about 80 years earlier.

The fissures below water level in old puddle cores might be attributed to that cause, and the problem seemed worthy of further investigation.

The Authors, in reply, wished to thank Mr Fairbairn for correcting a bibliographical error, and noted his suggestion that sodium ions, and not potassium ions, were responsible for the effect described by Drouhin, Gautier, and Dervieux. If that was so, it was important, since sodium was far more commonly present in natural waters than was potassium.

Mr Bryan's statement was particularly interesting since it showed that the phenomenom of base exchange occurred in nature on a large scale, and that the fact was well known to water supply engineers. That that was so had undoubtedly been overlooked in the past by students of soil mechanics, but it was evident that it could be important since Mr Ward considered it possible that a failure of a dam was due to that cause. Clearly the effects of base exchange on earth structures should not be overlooked in any site investigations concerned with the use of clay fills on a large scale.

Before the ideas put forward by Messrs Fairbairn and Ward could be accepted, further experimental work was necessary, and it was hoped that it would, in due course, be undertaken.

Mr Craven's suggestion that numerous landslips in the valleys of the

[1] D. G. Davies, " The Harrogate Dam Failure." J. Instn Wat. Engrs, vol. 7, p. 57 (Feb. 1953).

Pennines might have been caused by rapid draw-down conditions set up by the draining of glacial lakes, was an excellent example of the application of soil-mechanics principles to geology.

Mr Jakobson had given further information regarding loading berms which was very valuable and should be used in conjunction with his Paper in "*Géotechnique*" in 1948. The Authors agreed that recent work using the vane test had demonstrated that, in fact, most recent clay deposits showed an increase in shear strength with depth, but Mr Jakobson's Paper would still be of value since it applied to the limiting case. His observations to the effect that loading berms increased settlement were important and should be remembered by all engineers proposing to use that method.

The Authors were grateful for Mr Jakobson's account of Swedish practice in relation to symmetrical displacement and for more recent information on the effect of bog blasting at Lake Aspen. The Paper quoted by them in connexion with the latter had been written a comparatively short time after the completion of the work and gave the impression that there had been no serious settlement after bog blasting. Mr Jakobson's data showed that in fact that was not so, and that considerable settlements had taken place in the ensuing 7 years, although equilibrium would now appear to have been reached.

Mr Fox had evidently given much thought to the problem of accelerated consolidation, but the Authors were not in entire agreement with all the points raised by him. The saving in the cost of filling was only one of several advantages claimed for that construction method, and was not the most important. The Authors considered the most valuable effect to be the reduction of the time required for consolidation, and for that reason they regarded the use of deep sand drains under the fills of the New Jersey Turnpike to be entirely justified. Observations had shown that movements were significantly complete before any paving was placed, thereby eliminating any costly maintenance. The considerable depths of the drains was made necessary by (*a*) the great thickness of soft deposits—up to 250 feet; (*b*) the width of the embankment—roughly 300 feet; and (*c*) the height—about 50 feet.

The method of dealing with weak deposits should be selected from a consideration of what it was necessary to achieve—dictated by the function of the structure—rather than on the basis of relative costs. The Authors did not agree that sand drains would be used to accelerate the consolidation of the "weakest upper strata of the subsoil." As explained by Mr Jakobson, side berms offered no solution where settlements could not be accepted, nor did they accelerate movement. Such shallow soft strata as envisaged by Mr Fox would undoubtedly be excavated and backfilled. So far as Great Britain was concerned, marsh areas frequently showed stiff surface crusts formed as a result of drainage and plant growth since the time of reclamation. That condition would not, of course, apply to unreclaimed saltings.

Mr Fox's statement that drains modified the stress distribution under the foundation was thought-provoking. The Authors were not aware of any practical experience which supported the idea and were not able to reconcile the values quoted in the example for settlements of the steel tank. In any case, those values only referred to consolidation settlements, and estimations of fill requirements based upon those movements were not strictly comparable, since no allowances were made for the material required by virtue of the plastic flow of the underlying soil. Further, was such a large differential settlement of nearly 2 feet tolerable, even for such a flexible structure as a steel tank ? Mr Fox made no mention of the time factor, and the Authors wondered whether the increase in the rate of settlement associated with sand drains would not have been well worth the additional expense of extra filling.

Referring to Mr Fox's comparison of the Swedish wick drains with sand drains, it had been pointed out in the Paper that while the Swedish method worked very well, the special machine required for installing the wicks was very expensive. Sand piles, on the other hand, could be placed using ordinary piling plant, and so did not involve any investment in machines which might only be used on infrequent occasions. It was also felt that the sand piles were less susceptible to damage than the wicks.

The Authors confessed that they had no simple methods to offer for determining in-situ permeabilities of soft fine-grained soils, and felt that the observation of trial banks was the most practical method of assessing the value of sand-drain installations. However, experience with ground-water lowering in sands had indicated that pumping tests, coupled with detailed site investigation, did enable quite reliable estimates to be made of the in-situ permeabilities of those soils.

The Authors were doubtful whether Mr Fox's proposed method of stabilizing banks by constructing ditches filled with granular material was practicable. To add any significant strength, they would have to be very wide and comparatively deep, since otherwise slip circles might develop at increased depths and pass under them. In shallow deposits of clay it might be possible by carrying out the same amount of excavation under the bank, to guard against both slope failure and excessive consolidation. The construction of the ditches, even if they succeeded in preventing slope failures, would not effect any reduction of consolidation settlement.

To sum up, the Authors held the opinion very strongly that vertical sand drainage was probably the best and most economical method of dealing with soft foundations, when road embankments had to be carried across great depths of compressible soil and where settlements after construction could not be tolerated.

Correspondence on the foregoing Paper is now closed and no further contributions will be accepted.—Sec. I.C.E.

AIRPORT ENGINEERING DIVISION MEETING

30 December 1952

Sir Hubert Walker, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Airport Paper No. 22

"The Reconciliation of Civil Engineering Problems with International Standards in the Siting and Planning of Airports"

by

Archibald Shaw Maclaren, M.A., M.I.C.E.

SYNOPSIS

Certain of the Standards and Recommended Practices of the International Civil Aviation Organization with regard to airport design are examined in the Paper, and suggestions are made both for an improved basis of internationally published data and for other information which it is thought should be included in the Standards.

Using the new Dar es Salaam airport and the airport project at Embakasi, Nairobi, as illustrative examples, the Paper also emphasizes the importance of a critical application of the Standards in the light of other aeronautical knowledge in order to meet to the best advantage the financial and engineering problems encountered in airport construction. The importance of making full use of wind data is stressed, and factors affecting the length of runways and over-runs and the width of runways and taxiways are discussed, together with questions of approach lighting, approach slopes, and certain constructional and health problems.

In general, the conclusion is reached that there is scope for important economies in international airport design, particularly at sites in the tropics.

Introduction

It may properly be said that, during the preparatory stages of a civil engineering project, the function of the engineer is to plan as well as to design, and to do both with a view to the maximum economy of cost consistent with fitness for purpose. Not least amongst the factors which together constitute fitness for purpose are utility, safety, and amenity. Those engineers who have been concerned with civilian airports during recent years have had available to guide them, as regards the first two of these factors, the Standards and Recommended Practices of the AGA Division * of the International Civil Aviation Organization and, in the case of health measures at tropical sites, the International Sanitary Regulations of the World Health Organization.

* Aerodrome, Air Routes, and Ground Aids Division.

It is perhaps a fitting tribute to those who prepared the International Standards and Recommended Practices and to those who have applied them in practice, that it is difficult to cite any case of an accident on a modern civil airport, involving life and limb, that can be ascribed to a deficiency in the siting or planning of the airport.* However, the I.C.A.O., in making the interim issue of its Standards and Recommended Practices for the design of airports in 1947, specifically warned the airport designer not to use those Standards and Recommended Practices as his sole guide, but rather to interpret them in the light of the most recent developments and trends in the aeronautical arts, modifying his design, where necessary, to reflect the most recent findings in each art.

Whilst recognizing that the I.C.A.O. Standards and Recommended Practices, properly applied to the siting and planning of an airport, have hitherto given complete safety from the operational point of view, the Author cannot but regard it as a pity that the wording referred to in the preceding paragraph has not been repeated in the more recently published Annex 14 of I.C.A.O., entitled "Aerodromes." Not only is there, at the time of writing this Paper, a difference between some of the findings of the AGA Division as presented in Annex 14 and some of the separate recommendations of the Airworthiness and Operations Divisions of I.C.A.O. (owing to the time-lag required for the international digestion, assimilation, and publication of new ideas in a developing industry), but there is also a risk, in the Author's opinion, that certain of the International Standards may, if applied without sufficient reference to other aeronautical knowledge, militate against both the maximum efficiency of design and the economy of construction which is becoming increasingly important everywhere today, particularly in the less developed countries of limited financial resources for whose betterment air transport is a vital necessity.

Whilst the presently differing requirements of the separate Divisions of I.C.A.O. may be reconciled during the next meeting of the AGA Division, scheduled for October 1952, the immediate effect is that certain dimensional aspects of airport design, previously fairly clear-cut, have become temporarily nebulous; there has been, therefore, a tendency to fix the leading dimensions of a trunk-route airport in terms of a known "maximum-requirement" aircraft which will use it in the immediate future, with a wide margin of provision for enlargement. In a recent case, to which reference is made later, the aircraft selected as the criterion were the Comet 1 and the future more powerfully engined Comet 2. The more rigorous requirements of these first commercial jet-engined aircraft—it should be remembered that they are only the first—and the differences within I.C.A.O. requirements may, if taken together, present an uncertain and, therefore, disturbing forecast of its commitments to any authority embarking either on the extension of an existing airport or on the provision

* One recent accident in which aerodrome design was at least a contributory factor is described in the Appendix.

of a new one, as well as to the engineers whose lot it is to advise. There is also, to the Author's way of thinking, a danger inherent in designing for the needs of a particular aircraft rather than for those of a class or group. However, beyond noting the particular effects on airport design which may ensue from the reconciliation of the different publications of I.C.A.O., and making some tentative suggestions for an improved basis of promulgation of international aerodrome requirements in the future, the Author proposes to regard these present uncertainties as being beyond the scope of this Paper. He would like, nevertheless, to express a hope that the present fairly wide field of reconciliation that is necessary between the financial and civil engineering problems of airport construction and the International Standards will, by appropriate action and agreement on the part of those concerned with the latter, be narrowed rather than increased in the future.

The Importance of Wind Data in Siting and Planning

One of the most potent of the considerations that influence the search for and selection of a site for an airport, and also the cost of construction on the chosen site, is the number and orientation of the runways which need to be provided for the safe and convenient operation of aircraft, together with their length.

In 1948, Sir Alexander Gibb & Partners and the Author were instructed by the Government of Tanganyika to report on the feasibility of constructing a new airport at Mtwara (Mikindani), on the East African coast, 250 miles south of Dar es Salaam. The new airport was intended to replace the existing airstrip, which would accommodate only light aircraft; this strip was of an indifferent quality, and would be eliminated if town-planning development took place around the port that was under construction to serve the southern ground-nut area. As not infrequently happens in undeveloped countries, no wind records were available for any place nearer than Lindi—about 40 miles to the north-west—and the East African Meteorological Department advised that these should not be taken as a basis because Lindi Creek was believed to exert a pronounced effect on the winds, which would not be the case at Mtwara. In addition, there were only two sites for a new airport that might be suitable topographically, and they were of limited extent, so that whether either could be developed to meet requirements depended, *inter alia*, on the number and directions of the runways that would be necessary.

Since the general characteristics of the winds which occur on that coast are well known, it was decided, for the purposes of verification of the suitability of the sites only, to attempt to form an estimate of the nature of the winds to be expected at Mtwara, using as a basis those which were known to blow at Dar es Salaam, where records extended over a number

of years. This was done in conjunction with the East African Meteorological Department, as described below.

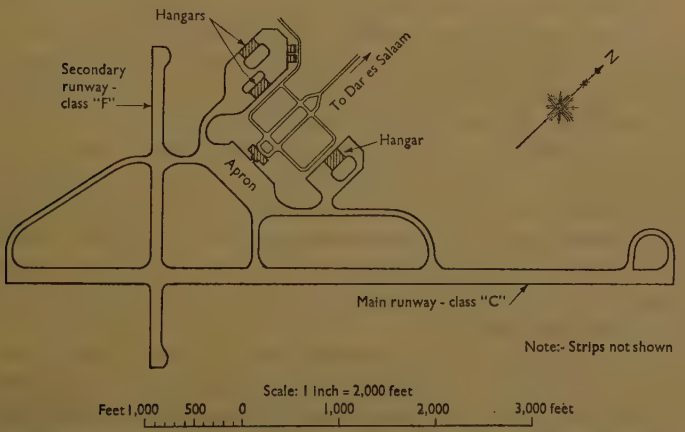
The winds on the coast are a combination of the north-east monsoon (October to March) and the south-east monsoon (April to September), blowing in wide arcs with their centres out in the Indian Ocean, and the daytime landward breeze or night seaward breeze, both of which blow at right angles to the average direction of the coast line. The monsoon winds become progressively weaker as they travel, and an examination of the records of coastal towns in Portuguese East Africa, further south, confirmed that the north-east monsoon would be weaker, and the south-east monsoon stronger, at Mtwara than at Dar es Salaam; in addition, since the periods of the monsoons depend on one overcoming the other, it seemed probable that the south-east monsoon would blow for a longer period at Mtwara. Owing to the general courses of the monsoons and the different orientations of the two coastlines, it seemed likely, also, that the directions of the winds at Mtwara would be anti-clockwise to those at Dar es Salaam.

Estimated quantitative and directional adjustments were made accordingly to the records of winds at Dar es Salaam, and the two monsoon estimates thus obtained for Mtwara were averaged in order to give an estimate of the winds there on an annual basis; the results indicated that two runways at most, at right angles to one another, would suffice for all operations, and that the main runway should be orientated within the sector 0 to 45 degrees, that is, north to north-east. Actual wind observations were commenced at Mtwara within a short time, and although the recorded frequencies of the occurrence of calms and of winds from eight directions during the first 12 months of observation differed from the estimated frequencies by an average of nearly 10 per cent and a maximum of 20 per cent of the total number of occurrences, the method of estimation, though approximate, was shown to have served its purpose of site selection; the orientation of the main runway as shown by the 12 months of wind observations was in the range 0—10 degrees. This method of approximate assessment of wind strengths and directions for the general purpose of site selection may prove to be of value at other places, particularly in the tropics and where the winds are generally fairly light and follow a pattern that is amenable to adjustment for locality.

It has sometimes been said that too close attention need not be given to a study of wind conditions during the siting and planning of an airport. However, the Author does not subscribe to this view, particularly in the tropics, where the prevalent winds may undergo marked diurnal and seasonal changes. In brief, the I.C.A.O. recommendation is that for as large a percentage of time as practicable, but for not less than 95 per cent, there shall be at least one runway which is capable of being used without the strength of the cross-wind upon it exceeding the value which has been selected as safe for the aircraft using the airport; and it has been common

practice when assessing the usability of an airport according to this principle, or when determining the number of runway directions that will be required to comply with it, to do so upon the basis of wind records extending over the 24 hours of the day and averaged over the year. It has also been noted in a recent Paper [1] that, owing to the increasing acceptance of higher cross-wind components than the hitherto standard 13 knots, the airport of the future may have only one runway direction, with perhaps another direction, or directions, for use by aircraft with low wing-loading if operational conditions permit. This principle has, in fact, been in use for some years.

Fig. 1



PROPOSED INITIAL LAY-OUT OF MBWALANI AIRPORT, DAR ES SALAAM

Fig. 1 shows, for example, the initial general lay-out projected for the new airport which is under construction for the Government of Tanganyika at Dar es Salaam. The main runway, orientated N.E.–S.W., is at present being constructed to I.C.A.O. Class " C " length, with provision for extension to Class " B " or, if necessary, Class " A "; while the secondary runway, at right angles to it, is projected initially to Class " F," also with provision for future extension. In determining this lay-out, which is permitted by the topographical and other conditions at the site, consideration was given both to the nature of the traffic expected to use the airport and to the day, night, and seasonal winds, which were analysed separately with the following results.

Under certain day conditions, and particularly during the period of the S.E. monsoon, one secondary runway orientated N.W.–S.E. was found

[1] W. J. Cozens and J. A. Glen, " The Influence of Ground-Handling Facilities on Airport Lay-out." Airport Paper No. 13, Instn Civ. Engrs, 1950.

to be necessary in addition to the main N.E.–S.W. runway, for use by those (usually smaller) aircraft that are more susceptible to the effect of cross-winds ; but it is not necessary at any time for the majority of trunk-route aircraft. Since the traffic foreseeable at the airport consists mainly of local-airline, private, and local-charter aircraft with only a few trunk-route aircraft, up to a total density which would not preclude switching operations from the secondary runway to the main runway for the movement of an occasional long-range aircraft, it is permissible to provide the secondary runway to the lower standard, thus achieving a considerable economy. In addition, only the main runway is necessary for all classes of traffic at night, so that a further economy is being effected by not lighting the secondary runway.

A separate examination of the night winds was also of assistance both in determining the best position for the traffic apron relative to the ends of the main runway, and in examining the most suitable arrangement of approach and taxiway lighting. Whilst the overall night-wind records gave no conclusive indication of a predominating direction of night operation on the main runway, a study of the hourly records disclosed that movements from S.W. to N.E. are likely to predominate up to midnight, when night flying is most frequent, and that the seaward breeze takes charge from midnight onwards only, when movements are very infrequent. Thus, the greatest need for lighting is for movements from S.W. to N.E. ; as regards apron location the use of 24-hour records would have given undue prominence to movements from N.E. to S.W.

It may also be noted in this example that the true need for, and the truer percentage use of, the secondary runway is given by the study of day-wind records only, the proved need being greater than if 24-hour records had been taken as the criterion. In an extreme case, indeed, the use of 24-hour wind records on an annual basis might give misleading results as to the need or otherwise for a secondary runway at all. For instance, if cross-winds of more than the acceptable strength occur on the main runway on 10 per cent of occasions during the day, and not at all at night, 24-hour wind records would indicate 95 per cent usability of the main runway and, therefore, no justifiable case for a secondary runway ; in fact, however, the daylight usability for the traffic most affected by cross-winds, which may well be the numerical majority of the traffic using the airport, would be nearer 90 per cent and a secondary runway would be highly desirable. Again while the annual mean usability of a single runway may appear to be adequate for all types of traffic likely to use it, the occurrence of strong cross-winds on it continuously during a season of the year may necessitate and justify the provision of a secondary runway for at least the lighter types of aircraft. This is the case already described at Dar es Salaam, but it is not the case at Nairobi ; the resulting difference of lay-out of the two airports may be seen by comparing *Figs 1* and *3*.

A close study of wind data may also help the designer to avoid adopting, without full justification for it, an orientation or lay-out necessitating unusually expensive construction work, and to use, instead, an arrangement which, while not endangering aeronautical safety or introducing more than a slight amount of inconvenience in operation, will achieve an economy in capital cost in this respect.

In all cases, therefore, whether for the adaptation of an existing airport or for the provision of a new one, the Author cannot stress too strongly the desirability of having adequate wind records for the actual site on which to base planning and design. When these are not available, or when they cannot be estimated with reasonable accuracy by the meteorological authorities, he would urge the taking of observations on the site by means of a Dines recording anemometer, or otherwise, for at least 12 months before the design is completed, and preferably for longer. In certain cases advantage may be gained by proceeding with the construction of a main runway on the basis of only 12 months of wind observations, allowing in the design and contract documents for the addition of one or more secondary runways after further observations have been made and studied.

In all studies of wind data the Author uses, in conjunction with the ordinary wind-roses, the vector-diagram method evolved by Marwick which was described and compared with his own earlier method during the discussion on a Paper by Cozens.[1] Fortuitously, a method similar to Marwick's has been published independently by C. S. Durst.[2] However, Durst's method, although basically the same as Marwick's, differs from it in that curves are introduced instead of the straight lines which in Marwick's diagram denote the limit of usability for each runway; this arises from the use of the Beaufort Scale of wind forces for plotting the wind circles, instead of an evenly progressive scale of knots or miles per hour and, in the Author's opinion, this militates against the convenient and accurate use of the method.

The Principal Dimensions of an Airport

Lengths of Runways and Over-Runs

At present the engineer who is asked to advise upon or execute the siting and planning of an airport intended to accept international air services, and who is not in a position to have ready access to aeronautical authorities for advice, is faced with many perplexities in the choice of the most economical runway length to meet the requirements of the anticipated traffic, both immediately and in the more distant future, which in aviation seems particularly obscure.

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[1] W. J. Cozens, "Problems in the Selection of Sites for Civil and Military Air Stations." Airport Paper No. 8, Instn Civ. Engrs, 1948. *Discussion* by Robert Marwick, p. 46.

[2] C. S. Durst, "Meteorology of Airfields." H.M.S.O., 1949.

Previously, the provisional recommendations of I.C.A.O. assigned ranges of the basic length of main runway to groups of aircraft in terms of their all-up operating weight. Whilst this method of selection was no doubt rough and ready, it did at least provide an initial guide that was of value in such matters as site selection and land reservation, where an incorrect early assessment of requirements may have serious consequences.

More recently, however, Annex 14 of I.C.A.O. leaves it to " the competent authority " to " select a main runway basic length to suit the requirements of the traffic that the aerodrome is intended to serve," and the classification of airports by letter in terms of runway length has become little more than a convenient basis for the recommendations on other airfield dimensions and on equipment.

I.C.A.O., in the Reports of its Regional Air Navigation Meetings, publishes the basic runway lengths required for take-off and for landing by aircraft types both in use and proposed for use in international air services, operating at their maximum permissible all-up weights, so that at first sight it would seem that the needs of the airport designer are met in this respect. However, there appear to be several weaknesses in the present system, which in the view of the Author are as follows :—

(*a*) There are no internationally agreed methods of calculating the factors by which the basic length of runway has to be multiplied to allow for the effects of, first, altitude, temperature and humidity, and secondly, longitudinal slope. Different countries use different methods.

(*b*) Nor is there, to the Author's knowledge, any collected and published data as to the different degrees to which these factors affect different types of aircraft. This may be of importance in the case of an airport designed to receive principally a particular type of traffic rather than mixed traffic generally.

(*c*) No break-down of the basic-length figures is given in the published I.C.A.O. documents. It would be of value to the skilled airport designer if, instead of the figures for basic runway length, or, better still, in augmentation of them, particulars were collated and published of actual aircraft performances under normal operating conditions (not test-pilot conditions or makers' pre-flight estimates), and if standard formulae were determined and agreed internationally by which the aircraft performance figures could be translated into the required basic length of runway, or of runway plus over-run, under both visual and instrument flight conditions. Such formulae would be of the following general type :—

Basic level length to be not less than :
X times the landing length under V.F.R. rules, or
Y „ „ „ „ „ I.F.R. rules, or

Z times unstick distance, with the provision that the total length must in any case not be less than either :

(i) the length for take-off to 50 feet in still air ; or

(ii) the accelerate-stop distance.

With these particulars before him, the designer could select the optimum basic length to suit a particular set of conditions with maximum economy.

In the case of aircraft still on the drawing-board or under pre-flight development, estimated performances published with a clear statement that they *are* pre-flight estimates, would be of value also.

(*d*) There is an inevitable time lag in the international collection and publication of data. Nor are the I.C.A.O. Regional Air Navigation Meetings held at very frequent intervals. Thus, in a rapidly developing art whose purpose is rapid transportation, there is an initial delay in making available basic information to those concerned with the provision of the ground facilities without which the aeroplanes cannot render their most useful service. To this delay there have to be added the further delays inherent in the preparation and consideration, financial and otherwise, of an airport project, and the time required for the preparation and letting of contracts, and completion of the works. The total time-lapse is considerable, indeed almost frightening in the light of the speed of advance of aviation, and any saving that can be made at any stage is most desirable.

In all this, it would be unfair to say that a few square yards of paving more or less, once the essential minimum is provided, do not matter very much to the aeronauts. The Author believes that I.C.A.O. has economy clearly in mind, and that strong views on cost are expressed by its member states. He believes, however, that still further economies are possible, and that civil engineers can be of assistance in showing how they can be achieved ; hence the foregoing proposals. Whether the published information on aircraft performances which he has suggested should be made available by I.C.A.O. or by the International Air Transport Association is a moot point. I.A.T.A. represents, in general, the consensus of operators' opinions to the I.C.A.O. meetings, but perhaps the promotion of traffic is more the direct concern of I.A.T.A. than of I.C.A.O., whose principal function appears to be that of regulation. It may be that I.A.T.A. could afford a useful ancillary service, in the interests of speed and up-to-dateness, by putting out interim statements for periodical ratification by I.C.A.O.

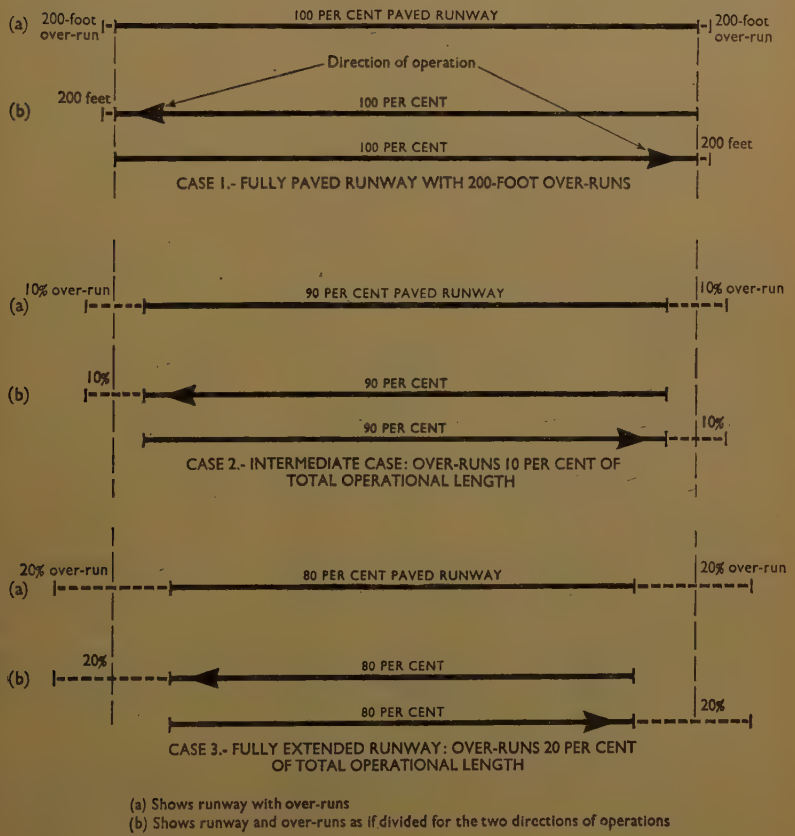
The extent to which prepared over-runs should or may be provided beyond the ends of the paved runway can also be, and frequently is, confusing to the non-aeronautically-minded controlling authority or engineer.

The matter is due for consideration by the AGA Division of I.C.A.O. in October 1952, but the only published recommendation at the time of writing is that each runway should be included in a strip, the length of which shall extend at least 200 feet beyond each end of the runway. However, the rising cost of construction (with the consequently greater need for capital economy), the increasing size of aircraft employed on trunk routes, the introduction of jet-engined aircraft, and the greater operating length required to comply with the new regulations of the AIR and OPS Divisions of I.C.A.O., have all recently brought about a reversion to an earlier principle. A length of paved runway is provided that is sufficient for regular and normal operation only, and an emergency safety margin is added at either end in the form of an over-run prepared to a load-bearing standard which is less than that of the runway. The length of each over-run may range, according to individual circumstances, from the minimum of 200 feet prescribed by I.C.A.O., to as much as 15 or 20 per cent of the total operational length, that is, the length of the paved runway plus one over-run. In other words, the length of paved surface may be reduced to as much as 85 or even 80 per cent of what it would otherwise be. This can be a very valuable economy, especially when the materials for paving have to be imported from a distance and locally available materials can be used for construction of the over-runs. (It should be noted, however, that additional ground space is necessary, equivalent to the length of one over-run.) The formulae proposed earlier for basic runway length in terms of aircraft performances should, therefore, state the requirements of both extreme cases—that of a paved runway with the minimum over-runs only, and that of a minimum length of paving with the maximum lengths of over-run. Any intermediate case may be interpolated to suit particular conditions as shown in *Fig. 2*, as, for instance, when the total length available on the site is limited to something less than that required for the maximum economy of paving.

Fig. 3 shows the initial lay-out of the proposed new airport at Embakasi, Nairobi, as an illustration of the use of over-runs. Only one runway is necessary to provide a very high coverage of all winds for all classes of aircraft but, owing to the physical altitude of more than 5,300 feet and the fact that the site is only about 100 miles from the Equator, the length-correction factor for a level runway is 1·55. In other words, the runway has to be 55 per cent longer than it would be under standard sea-level atmospheric conditions. Fortunately the site is open and, in the direction chosen, a length of 25,000 feet is available for the future, to include the runway, over-runs, and clearances for I.L.S. beacons. Initially, the length requirements have been based on the performance characteristics of the Comets 1 and 2, and a paved runway 10,000 feet long has been proposed, plus over-runs of 1,200 feet at either end. This proposal could not have been made, however, without technical assistance and advice

given both by the Ministry of Civil Aviation and by the British Overseas Airways Corporation as to the anticipated performance of the Comets. This stresses the need for the published data on aircraft performances already suggested.

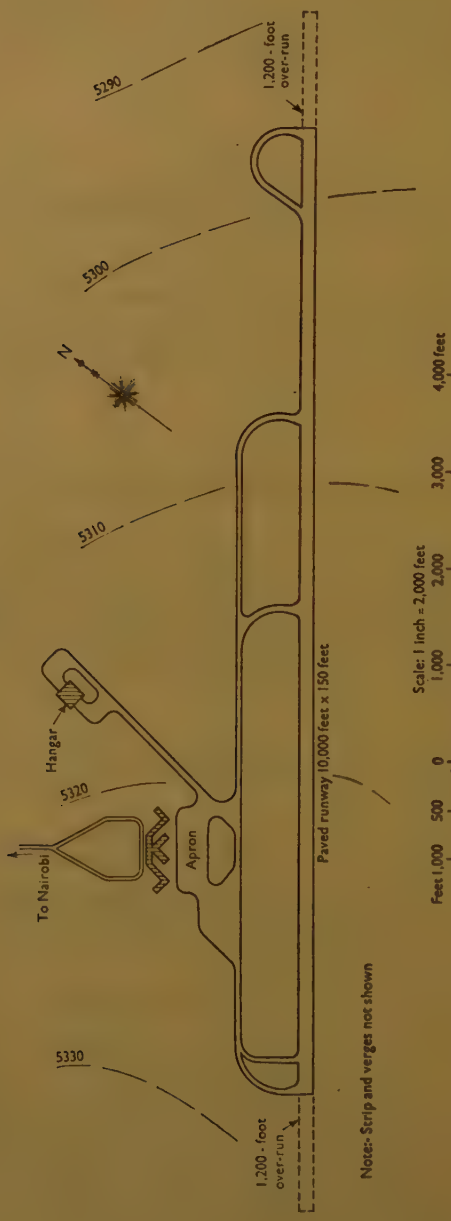
Fig. 2



(a) Shows runway with over-runs
(b) Shows runway and over-runs as if divided for the two directions of operations

The Author suggests that additional guidance is also required from I.C.A.O. as to the proportion which the length of a secondary runway should bear to the length of the main runway of the same class on the same airport, or as to an approved means of calculating this ratio in particular cases. A " round-figure " rule is given in Annex 14, but greater exactitude is desirable in the interest of economy of construction, and would also be useful in the selection of a site.

Fig. 3



INITIAL LAY-OUT PROPOSED FOR EMBAKASI AIRPORT, NAIROBI

Approach Lighting and Equipment

It is appropriate to refer briefly to approach lighting and equipment in connexion with the provision of over-runs. Cozens and Glen referred in their Paper to the obstruction of the over-run area by approach lighting of the line-and-crossbar type. Present British policy is to favour this type of approach lighting and to overcome the obstruction difficulty by using a very light type of light fitting carried on frangible mountings of aluminium-alloy tube, raised just high enough above the ground on a level site to clear the maximum stand of grass. Certain other countries, on the other hand, are believed to favour keeping the over-run areas entirely clear of any obstruction by lights. Thus the international position is fluid, and it would not be surprising if, as a result, a search were to be made for a new pattern of approach lighting which would be as efficient as the line-and-bar system, and which would, at the same time, provide a clear approach funnel.

As to the immediate present, there seems to be a strong case at sites where it is desired to use long over-runs and where the worst conditions of visibility are not so bad as to call for the maximum assistance from approach lighting, for using a " defined-funnel " type of lighting instead of the centre-line or centre-line-and-crossbar type. The expense, inconvenience, and, possibly, danger to traffic resulting from destruction of the whole or part of the lighting fittings by an " abnormal " aircraft-movement would thereby be avoided.

The I.L.S. transmitter and inner marker beacon are also potential obstructions in an over-run area. However, they are single isolated point obstructions and are not, therefore, so serious as a series of lights. In addition, the transmitter can be located beyond the end of the over-run on a suitable site, at the expense of a slightly increased spread of the beam on the runway ; and it may be that a marker beacon can be positioned in a small natural or artificial depression, although the Author has not actually seen this done and would be glad of information on the point.

Width of Runways

The I.C.A.O. Recommendations on the width of runways and on other leading airfield-dimensions are tied to the classification system of airports by letter according to the basic length of the main runway and the customary relationship with weight groups of aircraft, and they constitute a very valuable guide. However, the designer should give due consideration to them in the light of the particular conditions at any site, because :

(*a*) these dimensions are not strictly, and in some cases are not at all, functions of basic runway length ; and

(*b*) many of the recommendations have remained unchanged since the days of P.I.C.A.O., and there is an economic danger

that they will keep on coming up like hardy perennials, irrespective of advances in the aeronautical art.

Particularly is this so in the case of runway and taxiway widths.

I.C.A.O. recommends the runway widths shown in Table 1.

TABLE 1

Class of airport	Basic length of main runway : feet	Minimum runway width : feet
A and B	7,000 or more	200
C, D, and E	4,200 to 7,000	150
F and G	3,000 to 4,200	100

However, runway width is not a function of the basic runway lengths to which the different airport classes refer, and the important changes from 100 to 150 feet and from 150 to 200 feet have no justification solely on this basis. On instrument runways the spread of the landing beam makes the width a function of the actual length of runway plus the distance out to the location beacon (although in practice, as distinct from theory, a pilot will probably not wander to the extreme edge of the beam *and* persist in his attempt to land) ; thus a Class C runway at a high altitude may be in greater need of a 200-foot width than a Class B runway at sea level. Other factors which should influence the choice of width are :

(i) the prevalence and strength of cross-winds ;
(ii) the extent and nature of the runway verges ; and
(iii) the lateral spacing of the runway lights. This has been virtually standardized at 150 feet, and I.C.A.O. recommend that it should not exceed 180 feet ; otherwise a " dark lane " effect is presented to the pilot on landing.

Where the need for a greater width of runway rather than the lesser appears to be marginal, or where, for instance, it is occasioned by strong cross-winds which blow at certain seasons only, consideration may be given to providing the additional width in the form of strengthened verges rather than as an extension of the pavement, sometimes with considerable economy. By siting the longitudinal side drains of the runway outside the verges, provision may be made for an ultimate widening of the pavement to the greater width without too great a disturbance to the existing works. *Fig. 4* shows diagrammatically the proposal made in the case of Embakasi Airport, Nairobi ; a 150-foot wide runway is proposed initially, with two 25-foot verges, the main drain being positioned beyond

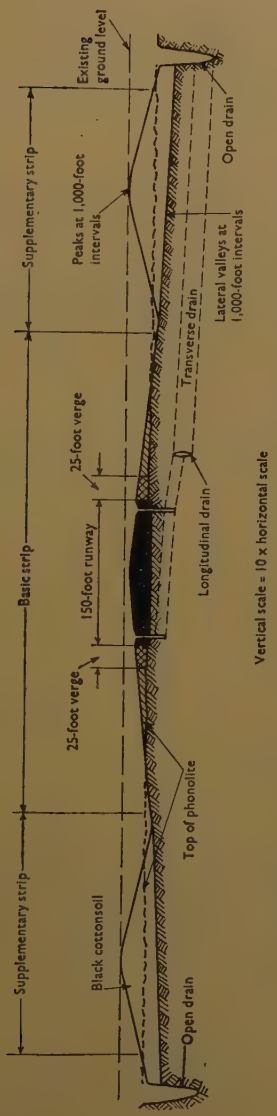


Fig. 4

PROPOSED CROSS-SECTION OF RUNWAY AND STRIP, EMBAKASI AIRPORT, NAIROBI

one of the verges so as to facilitate the ultimate widening of the pavement to 200 feet.

A considerable field for the use of judgement in the selection of the most appropriate runway width is thus left to the designer by the International Standards as they exist. However, the Author feels most strongly that, in the interest of economy, and despite the psychological need for the simplest possible standardization, a greater range of minimum widths should be accepted internationally than is given by the present 50-foot increases from 100 feet to 150 feet and 200 feet.

Consider a case such as the future widening of a 10,000-foot runway, as proposed for Nairobi. Each 15 feet added to the 150-foot width is equivalent in area and cost to increasing the length by 10 per cent, that is, by 1,000 feet! Table 2 shows the percentage additions of superficial area of paving corresponding to various increases of width, up to a maximum of 50 feet, in the case of a 150-foot-wide runway of any length.

TABLE 2

Increase in width : feet	Percentage increase in superficial area of paving
10	6·67
20	13·33
25	16·67
30	20
40	26·67
50	33·33

It seems reasonable to suppose that each of these percentage increases of superficial area, which are also percentage increases of width, produces a corresponding increase in operation safety, and, in the Author's opinion, runway widths should be standardized in intervals of 25 feet at most, and preferably in lesser intervals.

Width of Taxiways

I.C.A.O. recommends that taxiways should have the minimum widths shown in Table 3.

Whilst the figures quoted in Table 3 constitute a generally useful guide, the width of the taxiways on an airport is not directly related to the basic length of runway, except in a very broad sense by way of the undercarriage track of aircraft, which varies with the weight groups associated with the

TABLE 3

Class of airport	Minimum width of taxiway : feet
A	100
B & C	75
D	60
E	50
F & G	40

different airport classifications. Other factors which should influence the designer's choice of width are :—

(i) *The steerability of the aircraft expected as traffic*
With regard to steerability, nose-wheel aircraft can be handled more safely on a narrower taxiway than can tail-wheel aircraft.*

(ii) *The visibility conditions at the site*
Poor or bad visibility conditions on the ground increase the chances of an aircraft departing from the centre of a taxiway ; in general, therefore, a wider taxiway is more necessary under temperate conditions, or where fog and mist can occur, than under the relatively clear conditions of visibility which obtain for a great part of the time at many airports in the tropics.

(iii) *The nature of the taxiway verges*
As in the case of runways, the designer may consider whether he can provide an additional safety margin of width for the taxiways in the form of strengthened verges, rather than as an extension of the paving. The associated drainage problem is usually simpler than in the case of a runway.

(iv) *Dust and grit*
Particularly in the case of low-wing propeller-type aircraft

* When nose-wheel aircraft were still in their comparative infancy the Author suggested in his Paper " The Design of Land Airports for Medium and Long-distance Civil Air Transport," J. Instn Civ. Engrs, vol. 23, p. 100 (Jan. 1945), that the width of taxiways should be from 2 to 2·5 times the undercarriage track. P. H. Watson in his Paper " Aircraft Design with Special Reference to the Restrictions Imposed by Considerations of Airfield Design," J. Instn Civ. Engrs, vol. 32, p. 466 (Mar. 1949), considered that widths of 2 and 1·5 times the undercarriage track would be more correct for tail-wheel and nose-wheel aircraft respectively, although experience was insufficient to confirm the latter figure. More data seem to be needed before the full value of this consideration can be realized.

a nuisance can be caused by dust and grit being sucked up from the ground by the propellers when the engines are opened up for starting the aircraft rolling, for acceleration, or for making a turn. There is also, in such cases, an element of risk of damage to the propellers and/or engines, by flying grit or stones. Therefore the Author considers that on sites where the ground beyond the width of the taxiways, or of the taxiways plus prepared verges, is of such a nature that it might cause this trouble and cannot be rendered " inert " by the planting of grass, oiling, or other means, care should be taken to ensure that the total width of prepared surface is at least a few feet more than the greatest span of the outboard engines of any aircraft likely to use the taxiway.

In general, the earlier recommendations on taxiway widths appear to have " played safe," and it seems that the present I.C.A.O. recommended widths may be examined very critically in the increasingly revealing light of the advancing aeronautical art, and that they may be reduced in many cases, particularly at inland sites in the tropics.

Approach Slopes and Clearances

One effect of the new recommendations of the AIR and OPS Divisions of I.C.A.O. is that transport aircraft should have a minimum climb rate, with one engine cut out, of 1 in 66. This is a flatter slope than the flattest prescribed by the AGA Division of I.C.A.O. for the flight clearance planes beyond the ends of runways, which is 1 in 50. Thus, if aircraft designers were to adhere strictly to the AIR–OPS requirements, and airport constructors to those of the AGA Division, an event might occur that would be beyond the ability of anybody to reconcile !

In addition, jet-engined aircraft, for reasons of fuel economy, need to make a straight-in approach to an airport from their flying route, rather than circle the airfield or make the familiar " figure-8 " instrument approach ; and they tend towards a considerably flatter final approach to the runway than do piston-engined aircraft. For Royal Air Force purposes too, the approach slope prescribed for V.H.B. (Very Heavy Bomber) and all jet work is 1 in 100.

For all these reasons, the Author would advocate providing wherever possible, for the main and instrument runways of major civil airports, a clear approach and safeguarding of the airport environs in line with the runways at a slope of 1 in 100, particularly at any site within the hot and comparatively windless tropical belt, where, fortunately, it may sometimes be done without invalidating existing constructions or foreseeable development plans. Such a proposal has been made in the case of Embakasi, Nairobi.

Other I.C.A.O. Recommended Dimensions

The other I.C.A.O. recommended dimensions do not in general affect cost, but do affect operational convenience and safety, and they facilitate future expansion of the airport. When the land available permits, therefore, the designer can select with confidence the larger of two dimensions whenever there is any doubt, and by doing so he will also have a comfortable feeling that he has taken out a reasonably priced insurance policy against unknown future developments.

Constructional Problems

Runway and Taxiway Verges, and Over-Runs

Little or no guidance is given to the designer by the International Standards as to the desirable lateral extent of runway and taxiway verges, or as to the degree of bearing capacity to which they and over-runs should be prepared in comparison with the strength of the runway. The sole recommendation of I.C.A.O. Annex 14 on the subject is :—

> " Those portions of the strip immediately beyond the limits of the runway should be so prepared or constructed as to minimize hazards, arising from differences in load bearing capacity, to an aircraft running off the runway."

The problem of strength is, of course, intimately linked with questions of cost and of the materials and construction equipment available, and it varies with each site, but the Author considers that a very useful purpose would be served if I.C.A.O. were to indicate in its future publications :—

(i) Guiding principles for the selection of the width of additions to paved runways and taxiways, in the form of strengthened verges, in different circumstances.

(ii) A desirable range of bearing capacity, expressed as a percentage of that of the paved runway, for :

(*a*) runway over-runs, and
(*b*) runway and taxiway verges.

Merely from having flown many thousands of miles as a passenger, the Author feels that whilst the pilots of aircraft would naturally like to have both over-runs and verges as strong as possible, they would; if confronted with a clear choice between the two, express the opinion that they would rather have the verges stronger than the over-runs. Possibly some data on this question might be compiled from statistics of " abnormal " operations.

Grading and Treatment of Strips

The strip within which an instrument runway is enclosed is compounded in essence of a " basic " strip sandwiched between two " supplementary " strips. The strip widths recommended by I.C.A.O. are 1,000 feet for a basic-plus-supplementary, in other words a total instrument

strip at any airport; 700 feet for a non-instrument strip in the case of airport Classes A, B, and C; and 500 feet for Classes D, E, F, and G.

Annex 14 also makes certain recommendations about the grading of strips and adds a footnote:—

> "It is not intended that the portions of the strip beyond a distance of 75 metres (250 feet)* from the centre-line of the runway be necessarily prepared to the same extent as the portions within this distance, but rather that they be clear of hazards to aircraft."

An interesting problem in this connexion arose during the preliminary design of the proposed runway and strip at Embakasi, Nairobi (*Fig. 4*). The site is covered with a layer of black cotton soil to an average depth of about 27 inches, overlying decomposed phonolite which in turn covers the harder rock. It is necessary completely to remove the black cotton soil under the runway in order to obtain an adequate foundation, this material becoming exceedingly treacherous when wet; on the other hand, sufficient slope for drainage of surface water has to be provided over the total strip width of 1,000 feet. Were the drainage slopes to be provided as continuous outward falls from the runway, either the outer edges of the "supplementary" strips would have to be excavated into the harder material below the black cotton soil, or the total thickness of construction of the runway courses would be greater than is necessitated by considerations of strength alone. Either alternative would add cost, and the solution shown in *Fig. 4* was proposed as a compromise. Briefly, the surface water is to be carried away in longitudinal open drains located beyond the outer edges of the total instrument strip; and, while the two halves of the "basic" strip are graded uniformly outwards, the "supplementary" strips are graded transversely as humps and longitudinally as a series of low watersheds and drainage valleys (the latter forming extensions of the slopes of the "basic" strip), arranged at 1,000-foot centres, crest to crest. A certain amount of ponding is expected at the outer edges of the "basic" strip during heavy storms, but only to an extent that is not considered detrimental to flying operations.

Another point that has been discussed in connexion with this airport proposal is the purpose of the "supplementary" strips. Usually the view is taken that the word "hazards" in the footnote in Annex 14 referred to above means "flying hazards"—in other words, that the "supplementary" strip width is provided as an increased width of flying lane clear of obstructions to flight above normal ground level. In some cases, indeed, low hedges have been permitted within this area. However, certain aero-

* This figure does not tally with the 350-foot half-width of the non-instrument strip of Classes A, B, and C; these non-instrument strips, therefore, become a 500-foot-wide "basic" strip plus two 100-foot-wide "supplementary" strips. The Author has wondered whether the figure should not be 350 feet for these Classes of airport.

nautical circles contend that the supplementary strips should also provide an area suitable for a " belly landing " in the event of undercarriage failure. Owing to the happily infrequent occurrence of such an operation, it seems difficult to justify any special expenditure designed to improve the supplementary strips for the purpose, and the Author would suggest only that clarification by I.C.A.O. of the purpose of the supplementary strips might save discussions between engineers and their employers or clients in the matter !

No other constructional problems have presented themselves to the Author which require reconciliation between their civil engineering aspects and the International Standards. Rather are they to be regarded as the normal problems encountered by the engineer when complying with a set of design criteria.

Health Measures

Under the International Sanitary Regulations, every sanitary airport situated within a yellow-fever endemic zone must be freed from mosquitoes by systematically destroying them in their larval and adult stages both within the perimeter of the airport (defined as a line containing the airport buildings and any areas used or intended to be used for the parking of aircraft) and within a protective area, commonly known as the anti-amaryl zone, extending, free of buildings, for a distance of 400 metres (or 440 yards) around the airport perimeter. Beyond this zone again, the suppression of malarial mosquitoes should be undertaken, when they exist, although this is not a part of the International Regulations.

At certain sites these precautions necessitate the clearance of bush, native dwellings, etc., and drainage works, the cost of which has to be suitably debited. A case arose in the Author's experience, in East Africa, in which the total cost of these works was at first regarded as a direct charge against the airport, but it was realized later that the improvement, from the sanitary point of view, of a large portion of the area would benefit also certain lands which were designated for future town and industrial expansion. It was thus possible to agree to a sharing of the cost between the different forms of development, with a consequent reduction in the total cost of the airport.

In the Author's opinion, no other attempt whatsoever should be made to compromise on the international health requirements without the best, and authoritative, medical advice ; a spread of an outbreak of yellow fever can have results infinitely more dire in terms of human loss and suffering than can an aircraft accident, such as might result from an ill-judged attempt to juggle with the international standards of technical design of the airport.

Conclusions

Throughout all the stages of conception, siting, planning, and design of an international airport, a careful and critical study of the International

Standards and an intelligent interpretation of them, in the light both of the engineering problems arising and of the most recent knowledge of the aeronautical arts, is of immense value; there is also, in the Author's opinion, room for improvement both in the Standards themselves and in the frequency of their revision and publication in the interest both of operational efficiency and of constructional economy.

Not only may economies such as have been indicated in this Paper mean that more money may sometimes be devoted to the other vital parts of an airport—aprons, buildings, lighting, radio aids, and ancillary equipment of all kinds—so that these need not be skimped, and a well-balanced traffic-handling unit may be achieved; but they also become of increasing importance as the size and load-bearing capacity of the airport under consideration increases, and also in cases where a larger-than-usual correction factor has to be applied to the basic length of the runways. In extreme cases indeed, they may, taken together, make the difference between a territory with only limited finance available being able to afford an up-to-date airport and its having to struggle along with existing limited facilities, both to its own detriment and to that of air communication generally.

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The Paper is accompanied by four sheets of diagrams from which the Figures in the text have been prepared, and by the following Appendix.

Appendix

At Philadelphia Airport, U.S.A., the steel airport-boundary fence was within the I.L.S. transmitter interference range. It was, therefore, depressed in a 10-foot-deep ditch, resembling a ha-ha, 279 feet from the end of the runway. At this distance, approach-slope clearance was probably also involved.

On 14th January, 1951, a D.C.4 of National Airlines landed too far down the runway after an instrument approach in poor visibility and the runway is believed to have been iced. The aircraft overshot the runway and caught fire on the far side of the ditch after hitting it. The fire tenders could not reach it properly because of the ditch and there was considerable loss of life. It is reasonable to suppose that, but for the ditch, which is now acknowledged to have been a misguided attempt to reconcile other requirements with design standards, the serious nature of the accident would have been lessened. Either,

(*a*) there might have been less damage to the aircraft and no fire, or,

(*b*) the fire might have been extinguished more quickly and with less loss of life.

Discussion

The Author, in introduction, said that he had been asked to draw attention to two points which he had not mentioned in the Paper.

The first was that, on undulating sites, it was often difficult to decide how far cost should be incurred in eliminating longitudinal changes of gradient along the runways. Although limiting gradients and rates of change of gradient were set out in the I.C.A.O. report, if they were strictly applied they sometimes produced unsatisfactory results from the pilot's point of view.

That had never come within the Author's experience on aerodromes which had been designed to the I.C.A.O. standards. Perhaps the answer to it lay in what he had tried to stress in other connexions in the Paper—that the designer should apply the standards with reference to the aeronautical arts, and, if he had not the knowledge to do that himself, he should refer his proposals to somebody in authority who had. For example, the early airstrip at Mtwara—referred to in the Paper—had been cut out of a sisal plantation with the minimum of regrading. After a time it had proved to be short for the planes that used it, so that they had had to touch down on the extreme end of the runway and, when landing into the south-east monsoon, they had touched down on a rising gradient. Quite soon after landing, the gradient reversed over a crest, and that either threw them into the air again or, if it did not actually do that, it introduced an unpleasant element into the landing process. Of course, in that case, the gradients had not been to I.C.A.O. standards, but it was an illustration of the kind of thing to avoid.

The second point was a question. How far could increased constructional costs be justifiably incurred in order to produce a convenient and compact runway and taxiway pattern? That problem again usually arose on undulating sites.

The Author did not think it was possible to generalize on that question. Each site presented an individual problem. The anticipated volume and nature of air traffic was usually the deciding factor in determining the extent or complexity of the lay-out to be provided and, generally speaking, the usual course was to provide at first only sufficient for the traffic expected during the next few years, within the framework of a master plan of ultimate possible development, more and more of which could be added as time went on and as increasing traffic created the need and warranted the expenditure. Of course, care should be taken that what was planned for the future could be built without interruption of the air traffic using what was there already.

Mr R. L. Fitt remarked that he would first like to endorse all that the Author had said about the various unknowns which had to be solved before

a modern aerodrome could begin to take shape on the drawing-board and in the field. In fact, since he had helped the Author a little in the preparation of the Paper he could not very well do other than agree with all that the Author had said. Ahead there lay uncharted territories of aircraft development, and the engineer felt a bond of sympathy with Christopher Columbus, of whom it had been said that when he set off to discover America he did not know where he was going, when he got there he did not know where he was, and when he got back he did not know where he had been.

In facing those perplexities the engineer might take comfort from the fact that a similar state of affairs existed in the early days of rail and road development, and in the gradual evolution of water-borne traffic from small sailing craft to the present great ocean liners. In connexion with railways, one thought of the cost which the Great Western Railway had incurred in having to change its gauge, and of the similar problems which confronted such countries as Australia, India, and, particularly at the present time, the continent of Africa, as railway systems extended and needed linking up if traffic was to flow quickly and cheaply.

In the case of air transport, development was proceeding at a bewildering pace, and, as the Author had pointed out, the time-lag which inevitably resulted between the collecting together of information, its discussion at various international conferences, and subsequent issue in the form of official documents for the guidance of the engineer, was such that much of the information necessarily remained somewhat vague and nebulous and some of it was out of date before it became available for general use.

In common with the aircraft designer, the civil engineer—and it was true also of his mechanical and electrical colleagues—had two main considerations before him : the first was to ensure the safety of air travellers, and the second was to achieve maximum economy in costs. The growth of popularity of air travel had not suffered from accidents brought about by inadequate design of airports. The Author had quoted the only such accident which was known to him. Such few accidents as had occurred in or near airports had generally arisen from some error of judgement on the part of the pilot, or from failure of aircraft engines or equipment at take-off. Improvements in lighting and radio aids were continuing to increase the serviceability of airports at night and in adverse weather conditions, but those developments brought in their train associated problems of the proper width and length of runways and over-runs. Up to the present, however, it could not be said that safety had been sacrificed by the engineer in the service of economy.

With regard to cost, the engineer had to bear in mind not only the initial capital cost and the cost of maintenance but also the incremental costs which might occur as and when extensions to runways and taxiways became necessary to meet the demands of new and heavier types of aircraft. Here the airport engineer was on common ground with his col-

leagues who were concerned with the design of trunk-road systems. He had to ensure, so far as he could predict it, that the foundations which he had created in the initial development were of sufficient strength to carry any loading which might be called for in the future. That he could do in the case of flexible runways, for instance, by thickening the runway surfacing; but he could perform a valuable service in the initial stages, if funds permitted, if he could arrange for the major element of strength to rest in the base-course materials, which were in most cases available locally and at less ultimate cost to his employer.

By far the largest item in the cost of any new airport was that of the runways, and it would be of inestimable value if some limitation in overall basic length could be established and if aircraft designers could be called upon to work within those limits. Such a development had been forecast in a Note issued by the International Air Transport Association in February 1950, in which it had been pointed out that the cost of aerodrome production had risen at such a rapid rate that an arbitrary limitation of runway length might be imposed by I.C.A.O. or by individual States in the near future. It was felt that a basic length of 8,400 feet would not impede aircraft development and would not affect the operating weights of present-day aircraft, but would serve as a guide to aircraft manufacturers when designing new aircraft.

It could be appreciated that, at the present period of rapid development in the aircraft industry, it might be considered a retrograde step to set limits to the length of runway and to the unit loading on runway surfaces. On the other hand, the introduction of a new type of aircraft, with heavier wheel loadings and requiring longer lengths to take off and land, might mean a very heavy capital expenditure on airports throughout the world. It was a sobering thought that the list of land aerodromes required for regular use by international air services in the European-Mediterranean Region alone contained more than a hundred names.

The Author had very rightly emphasized those points and had drawn particular attention to the possibility of achieving economy, where site conditions permitted, by a reduction in the length of fully paved runways with increased length of over-run. He had also drawn attention to possible economies on the runway verges and in the preparation of the supplementary strips. It would be helpful, particularly in the case of a very long runway such as was required at the altitude of Nairobi, if some critical angle could be established with its apex at the I.L.S. localizer beacon within which it was considered essential to provide a fully paved runway. Possibly that critical angle could be varied to cover differing degrees of minimum visibility at individual airports and the maximum cross-winds which were likely to occur at those airports.

The Author had also drawn attention to the gradual flattening of approach and take-off angles. There were surely existing airports at which it would be very difficult, and even impossible, to reduce the slope

from 1 in 50 to 1 in 100. That slope was of great importance in siting airports, and if it were to be reduced to 1 in 100 the figure should be established as the irreducible minimum for all future aircraft operations. Unless airports were going to be sited at long distances from important centres of population, the sterilization of large areas of land would represent a very considerable incremental cost to the authorities owning the airports.

On p. 288 the Author had drawn attention to certain shortcomings in the published information on basic runway lengths for various aircraft types. In that connexion, Mr Fitt was very pleased to note that the European-Mediterranean Regional Group of I.C.A.O. had recently drawn attention to the fact that the tabulated characteristics of aircraft did not include sufficient information on take-off and landing lengths under standard conditions, and had recommended that that information be made available. Furthermore, they had noted that adherence to the code limits of Annex 14 might eventually lead to standardization of aerodromes and of the aircraft using them.

Mr J. A. Skinner observed that in the present age of standards there was an increasing demand for clear-cut requirements for airfields. The problem which had to be faced in the laying down of standards was that they had to apply universally and, so far as airfields were concerned, they had therefore to apply all over the world, to all climates and to all nations, whatever their particular engineering outlook. That last point was perhaps sometimes overlooked. At the same time, the standards had to be reasonably permanent if they were going to be of any real use, because otherwise airfields would become out of date as quickly in the future as they had in the past.

It would be natural to say that no standards for airfields should ordinarily be written in the absence of complete standards for the aircraft using those airfields. Such standards did not exist. Perhaps the aircraft designer did not wish to be so standard-bound as the engineer. Although he understood that I.C.A.O. laid down certain operating requirements for aircraft, they were by no means comprehensive and they did not in fact cover precisely such important requirements as the ability to land across the wind, a factor which he considered had an enormous bearing on the cost of airfields. It was also noted that, as the Author had mentioned, in certain cases specified aircraft performances did not in fact agree with the airfield standards so far as approaches were concerned; but Mr Skinner understood that that had been put right by I.C.A.O. at a meeting held during the autumn.

To his mind, the three most important factors in the economics of airfield construction were the number of runways, the strength of the runway, and the length of the runway. He felt that other factors, such as clearances and widths, were insignificant compared with those. Even the approach clearances became less important as orientation became less critical.

The main factor which would affect the siting, the cost, and even the

possibility of building an airfield in a particular location was the number of runways required. On the question of the number of runways, I.C.A.O. Standards were very non-committal. The subject was largely bound up with the unspecified cross-wind component, and the fact that that cross-wind component was, he understood, constantly going up meant that the one-runway airfield all over the world was getting nearer and nearer. Fifteen years ago it had been assumed that aircraft required a triangular system of runways, but now military aviation had gone so far in some cases as to accept a single-runway airfield. In between, there were the criss-cross parallel runways, as at London Airport, Catherine-wheels, stars, and a host of patterns which might in the future be regarded as representing only a passing phase.

The other essential standard demanded by the engineer was the strength to which the runways should be built. There again the question was very complicated, since throughout the world there was no agreed method of expressing the load-bearing characteristics of aircraft, particularly those with multi-wheel undercarriages. There was also the fact that the undercarriage design seemed to be constantly changing. The aircraft designer was obviously trying to help the engineer by spreading the load and putting more wheels on the aeroplane, but Mr Skinner noticed that tire pressures were being increased in order to make those wheels smaller so that they could be stowed away, which in effect made the aerodrome engineer's problem more difficult, since more attention had to be paid to the surfacing. Until aircraft loading and undercarriage design had become much more stabilized and established than they were at present, it would be very difficult for any international organization to lay down strength figures for use by the airfield engineer. A possible solution was, of course, to accept the British Load Classification Number system, but that had not as yet been accepted internationally, and that it had not been generally accepted was bound up with the fact that there were so many compromises when it came to a question of applying that system to the multi-wheel undercarriage loading.

The third of the factors to which he had referred was the length of the runway. That was very important, but, knowing the ever-changing characteristics of aircraft, it was difficult to see how firm standards could be laid down. The present I.C.A.O. provisions regarding length seemed to suit present aircraft only up to a point, and the Author's example of Nairobi was an instance where they had to be disregarded; the Author related the length to a particular aircraft and not to an I.C.A.O. Standard.

Mr Skinner suggested that I.C.A.O. might place more emphasis on extensibility, since, up to the present, lengths had always been increasing. The Author had planned for 25,000 feet. When the engineer was planning an airfield, he certainly had to take extensibility into account. It was significant that the need for the new Nairobi airfield was largely bound up with the fact that the runway on the present airfield could not be extended

any further, although in the 1930's, when the present airfield had been built, it had presumably been thought that there was plenty of room.

If airfield engineers were compelled to assess the length of runway from operating characteristics, as apparently the Author had done at Nairobi, it would be more useful if I.C.A.O. dealt not with nominal lengths but with the specification of such things as the safety factors to be allowed in addition to the actual take-off and landing lengths. It would then be possible to work out, from the characteristics produced by the aircraft manufacturers, the length of runway required. The I.C.A.O., on the other hand, certainly ought to give guidance on adjustments for altitude and temperature.

The three points which Mr Skinner had mentioned—the number of runways, the strength of the pavement, and the length and extensibility—were in his view by far the most important items on which guidance was required from the international standardization organization, and, as he had already said, he felt that most of the other factors were insignificant by comparison. He therefore considered that the engineer should say to the Standardization Committee that he required far more guidance on those three items. Before the Committee would be able to issue that guidance they would have to look far into the future and visualize possible aircraft trends. It was only by doing so that they would be able to standardize airfield construction throughout the world. If that were done, it might be that the aircraft development of the future would be guided by the airfields which existed for their operation.

Mr R. F. Lloyd-Jones said that in any discussion of airport design one was immediately faced with the fact that many of the problems involved did not lend themselves to precise evaluation. That was a fact which most engineers were reluctant to accept; nevertheless it had to be faced. For example, one of the most obvious things that had to be determined was the length of the runway. It was not a question of selecting a runway which would enable the aircraft to get into the air or to land, but of knowing what margin to allow over and above the minimum requirement to enable those aircraft to operate with an acceptable degree of safety. The length of a runway might first of all be determined by the theoretical or normal landing and take-off length required by a particular aircraft, but to that length margins had to be added to allow for malfunctioning of the aircraft or its engines, human errors, bad visibility causing inaccurate navigation, deficiencies of the runway surface, and so on.

Those margins were substantial but indeterminate, and, when they were added to the minimum requirement to get the aircraft into the air, it was somewhat pointless to have attempted to arrive at an exact distance in the first instance. Apart from that, it had to be borne in mind that the runway was probably going to last for two or three times the life of any aircraft of which it was possible to get exact knowledge at the present

time, so that in any case it was necessary to make an estimate of what was going to be wanted in the future. It might be that a different type of aircraft would operate from the airfield. In any case some allowance had to be made.

It was inevitable and proper that standards should be evolved. Obviously it was only if there was some uniformity throughout the world that pilots could operate, knowing what margins were allowed, and could use the various airports with confidence. There was the further advantage that standardization would tend to make aircraft designers conform to some limits. One important fact which had to be borne in mind, however, was that any dimensional standard implied a safety standard. So far as he knew, no one had suggested what the odds should be—that was to say, what the limiting chances of an accident occurring ought to be. He imagined that the passenger decided that in the long run, but he felt that the dimensional standards produced by I.C.A.O. had not been arrived at by any precise or actuarial calculation, but as being a compromise of opinions of people who had had experience of operating aircraft over a number of years.

One could not have a standard unless the conditions which gave rise to the risk of accident were the same at all airports. If the conditions at one airport differed from those at another it was futile to provide a standard, which would only result in extravagance at some airports or dangerous risks at others, so that inevitably there would be certain factors in the design of any airport which were indefinite. It was with that in mind that he was not entirely persuaded by the Author that the runway length could be standardized by taking the performance of known aircraft and multiplying those lengths by some arbitrary standard factor. It might be that some such formulation would permit runways to be calculated to conform to a standard within very close limits, but that was not what was sought to be achieved ; what was wanted was a runway which would provide a degree of safety which was acceptable.

The conditions which gave rise to accidents varied considerably from airport to airport. An airport situated amongst mountains or hills, where there were turbulent air conditions or where the pilot had to approach over high ground, usually meant that it was necessary to allow a rather longer runway ; and there were other conditions. If one were to try to arrive precisely at the length of a runway one ought to consider not only the performance of an aircraft but its reliability, whether the operating companies at an airport maintained their aircraft satisfactorily and what standard of training they had for their workers. He thought that it was possible to arrive at a satisfactory safety position for runways only by leaving the decision to some expert or someone who had sufficient experience of similar operations elsewhere to exercise his judgement in deciding what the length should be, after making whatever allowances were necessary for the particular conditions which existed at the site in question.

There was another point which sometimes arose, and which he could perhaps illustrate by considering two hypothetical airports, both exactly the same and operating under exactly the same conditions, but where one of the airports was in frequent use and the other was used only occasionally. Was it reasonable to reduce the standards on the airport which was occasionally used ? Obviously the chance of an accident occurring during any given period of time would be greater at the busy airport, but that was not the point ; what one wanted was to have the same risk of an accident for only one movement, and, that being so, obviously the standards at the two airports had to be the same. It was possible to get round that issue by altering the limiting conditions under which the infrequently used airport would be operated, but the basic principle remained. He raised that point because in the Paper the Author had mentioned that at Dar es Salaam night landings were more frequent in one direction on the main runway than in the other, and had said that therefore the aviation lighting was more important in the frequently used direction. Mr Lloyd-Jones was not quite sure what was implied there, but he wondered whether that meant that in fact there was any reduction of standard in the less frequently used direction.

He was interested in the Author's discussion of the possible economy of increasing the length of the over-runs and decreasing the length of the runway proper. Obviously the amount of saving in cost would often depend on the strength which was required in the over-run areas, and the Author had said that some guidance ought to be given; but Mr Lloyd-Jones would be interested to know whether the Author considered that the ratio between the strength of the over-run and that of the runway proper should be a fixed amount or whether it should vary with the extent to which the main runway was reduced. The Author had also mentioned that the runway could be reduced in length up to 20 per cent. Did he consider that that was the greatest reduction which could be made, and, if so, what reasons had he for fixing it at that figure ?

The Author had suggested that there should be a greater range of runway widths within the international standards, and, as he rightly pointed out, a small addition to the width could mean a big percentage addition to the area of paving. Mr Lloyd-Jones thought that that matter required careful consideration. The question of runway widths was again an indeterminate matter, but it had been argued that it was important for psychological reasons to have a standard width. It helped the pilot to judge his distance during approach and landing. The width of 150 feet, which was predominantly used nowadays, was a width which had been proved by experience to be satisfactory for most types of aircraft and to suit the spacing of the runway lights. It would be interesting to hear expressions of opinion on whether that argument justified what might be a fairly heavy cost.

Many of the worries of the airport designer arose not so much from the

international standards as from the lack of them. It was very worrying to the engineer who had been trained in an exact science, but, as the Author had said, the engineer had to decide whether he was going to be a designer of the airport *in toto* or whether he was simply going to wait until someone told him what the various dimensions should be and then proceed to build it. Those were two very different jobs. Mr Lloyd-Jones did not think that it was possible to be an efficient designer of an airport without studying the aviation problems and the navigational and other problems which arose. It could only be done by very close association between an engineer and an aeronautical expert, but it was obviously better if the engineer had a sound knowledge of the problems himself. Above all, he had to have a lively imagination to be able to see what the future was going to require, and he had to have faith in his own beliefs and judgement, because, unlike engineers in other fields, he often had to persuade people to spend large sums of money which it was very difficult for him to prove essential.

Mr E. H. Lewis-Dale remarked upon the Author's reference to changes in gradient. Until fairly recently there had been just such a hump at Prestwick Airport in Scotland. The main runway had been built during the war, and so far as Mr Lewis-Dale knew there had been no standard at that time which determined the matter. The hump had been noticed by the pilots of Liberator aircraft, but then Liberator aircraft ceased to use the airport and nothing more was noticed until a Stratocruiser only just escaped bumping itself badly. The hump had been ironed out, and the process had been troublesome, because the runway had to be kept open. In ironing out the hump an improvement had been made upon the standards laid down by I.C.A.O.

Sometimes, however, standardization could be troublesome. If there were two flat runways which intersected, and it was decided to surface only one of them, because the other did not look so bad or was not used so often, in order to satisfy the letter of the requirements it would be necessary to feather out the 3 inches of macadam or asphalt on the first runway over a distance of 186 feet on the second, which put it outside the bounds of practical policy.

He was very glad that Mr Skinner had raised the question of the absence of standards, because that brought up the subject of the strength of verges. With the absence of standards of strength it was very difficult to say that a verge or an over-run should have a relation to the runway strength, when nobody was quite certain to what standard the runway had been built. At present, I.C.A.O. standards gave some wheel loads which he believed were called single isolated wheel loads, but unfortunately they left the tire pressure to the competent authority, which meant the country in which the aerodrome was built, and that was no help to anyone.

He had found some difficulty about the question of the strength of verges, because at airports in Britain, and no doubt at many airports in other countries, verges and over-runs were at present grass. Possibly on

some clay subsoils, if they were well drained and had good turf, that might be satisfactory, but it was extremely difficult to say that grass had such-and-such a strength, and it would be interesting to know whether the Author had any views on how the strength of grass ought to be determined.

It seemed to him a little difficult to say that the bearing capacity of a verge or an over-run should be a certain fraction of the strength of the runway. Perhaps that was not what the Author meant, but Mr Lewis-Dale felt that perhaps one should lower the standard (because of the infrequency of use, because it was going to need less maintenance, or because it would still support the aircraft) rather than say that the actual strength need not be quite so much, because in fact the same aircraft were going to use the runway and the verge.

On p. 293 the Author had suggested that the frangible fittings on the line-and-crossbar approach-lighting system might be a source of danger and additional expense. Mr Lewis-Dale had been told that those fittings caused no damage to aircraft that struck and broke them, and he would suggest that the expense of replacing them might be of secondary importance if indeed the line-and-crossbar was the best system of approach lighting and far more effective than any other. After all, it was to be hoped that aircraft would not use the over-run and approach area very much for running on, and if a machine did knock a fitting occasionally it would not take very long to put it back again.

Mr W. M. Hargreaves referred to the I.C.A.O. meeting mentioned on p. 282. The Author had suggested that there were several items of co-ordination necessary between the various Divisions of I.C.A.O., and that was in fact exactly what had been done at the AGA Division meeting held in October last. Mr Hargreaves had been a delegate at that meeting and would like to deal with a few of the points which had been raised.

On p. 288 the question of correction factors for runway lengths was mentioned, and Mr Skinner had touched on the same point. On that issue the AGA Division had agreed on corrections for altitude and temperature. The figures which they had agreed had been averaged, of course, over a number of aircraft types and chosen as more or less the mean of those types. It was clearly impossible to consider both the high and the low figures ; a mean had to be taken between the two, and that had been done in relation to that correction.

The AGA Division had also agreed that over-runs, referred to on p. 290, should be provided in excess of the 200 feet which had been allowed previously in Annex 14 of the Convention. A greater length of over-run could now be provided, but of course there were certain limitations on it. That would be a tremendous help in catering for a critical aircraft at a particular site. The Division had tended to stall on the making of any suggestion of what the strength of those over-runs should be, because it was, as several speakers had pointed out, a difficult point to decide or on which to make recommendations. They said that it should be provided

with such a surface that aircraft could run on it without hazard. That left it very wide open.

For a long time there had been no agreed cross-wind component which would make it possible to work out the number of runways required to give a certain usability, but the AGA Division had agreed upon a figure which was higher than it had been previously, with the result that a single runway would now cater for most locations—a very desirable state of affairs.

Those were the main points on which the AGA Division had succeeded in producing information which would permit the co-ordination necessary between the various Divisions concerned.

On p. 293, the Author had suggested that there should be a closer scaling of runway widths, because of the economy which would be achieved thereby, and that was, of course, very important; great economy was achieved by every yard of runway which was taken off. Mr Lloyd-Jones had pointed out, however, that there was much to be said for having a standard width of 150 feet. That had really been the intention of I.C.A.O. in the original suggestion that there should be no more than two standard widths. In fact, there was a third which provided for the few aerodromes of the smallest class, but in the main the widths suggested were 150 feet and 200 feet. There was a great deal in the point that the standard width helped the perspective view of the pilot landing the plane; he became used to a certain width and it gave him a certain scale and dimensional characteristics to which he was accustomed, so assisting his approach and landing. Mr Hargreaves was not in favour, therefore, of increasing the number of standard widths.

On the question of approach lighting, the AGA Division had, in fact, agreed on an international standardization of the centre-line-and-crossbar type. The Author had mentioned that there was something to be said for a " defined-funnel " type of lighting, but that type was now definitely out of favour, and I.C.A.O. had taken the sensible step of standardizing on the British system, so that the point made by the Author with regard to frangible mountings on centre-line lights was a very real one and would have to be tackled. It had already been stated that it was being tackled by the use of very lightweight fittings which could be knocked over without damage to the aircraft. That view had been accepted by most of the delegates at the international meeting and was accepted in Great Britain. The British policy that centre-line lighting should continue right down to the threshold of the runway had been generally accepted throughout the world.

In the section of the Paper on approach slopes (p. 298) it appeared that there was some misconception, for the Author had quoted a gradient of 1 in 66 as being a recommendation of the AIR and OPS Divisions. That figure of 1 in 66 was, in fact, a misinterpretation of the information in the AIR and OPS Divisions' annex; the requirement was that an aircraft should be able to clear obstructions by 50 feet plus one-hundredth of the

distance of the obstruction from the take-off point, which was not quite 1 in 66, and the AIR and OPS Divisions permitted aircraft to do a slight turn to avoid an isolated obstruction.

On those approach-surface slopes, no alteration had been made to the 1 in 50 slope ; it remained 1 in 50 so far as the international agreement was concerned. The only concession which had been made was that in particular difficult locations, either because of temperature or because of altitude, some flattening of the slope should be considered in consultation with the airport authority and the airline operator.

Mr John Briscoe suggested that both in the Paper and in the discussion there had been too much emphasis on the word " standards." In the annexes of I.C.A.O., he observed, the standards formed a very small proportion of the material which was available to the aerodrome designer. By far the larger part covered recommended practices, and those were put in as a guide where a standard was too difficult to set up for use everywhere. At many sites, indeed, it was impossible, or impossibly expensive, to lay out an aerodrome or a runway if every recommended practice was to be strictly observed.

The Author's general theme seemed to be that the users should lay down what they required and then leave it to the airport engineer to get on with the job. In Mr Briscoe's view, it was only the user of the aerodrome who could say how far the recommended practices of I.C.A.O. need not be strictly followed in certain cases, and those users were the people with the operational experience, the pilots. He wished that the Author had suggested more consultation with the users.

In particular, the Author had made mention of the users supplying sets of figures and suggested that the International Air Transport Association should publish them and that I.C.A.O. should later " rubber-stamp " them. That showed a slight misconception of the functions of I.A.T.A. and I.C.A.O. Mr Briscoe gave an example of one of the difficulties in giving those figures. Jet aircraft such as the Comet, with very high fuel consumptions, could carry fuel up to nearly 50 per cent of their maximum gross weight at take-off, but had a very limited maximum landing weight (in the case of the Comet, 75 per cent of the maximum at which it could take off). It did not follow, therefore, that those aircraft were penalized if a runway was not long enough for them to take off at the full weight permitted by their certificate of airworthiness. On shorter flights, from Bahrein to Karachi, for instance, if the aircraft had taken off at its maximum gross weight it would arrive over its destination at a greater weight than that at which it was permitted to land, and, if it had made such a take-off, might have to dump thousands of gallons of valuable fuel before it could land. With jet aircraft, therefore, it was necessary to examine each stage from an aerodrome to see what take-off weights were required before it could be said what the take-off length required was for that particular aerodrome.

The Author had suggested that marker beacons might be placed in small natural or artificial depressions, and asked whether that had been done. There had been an instance in America where not a marker beacon but a fence had been sunk in an artificial depression so as not to cause obstruction to an I.L.S. beacon. Two winters ago, a Skymaster aircraft had over-run the runway and ended up in that ditch. The fire engines had been unable to get to the aircraft, which had been considerably damaged and had then caught fire, leading to considerable loss of life. It would seem, therefore, that any artificial depression was a considerable danger, and any such depression limited the amount of ground which could be used as over-run.

One point relating to economy in the design of aerodromes on which it would be interesting to hear further views was how much longer a runway should be made in aerodromes where high temperatures were involved, since such high temperatures occurred only at certain times of the day and in certain months. There had been a usability laid down on cross-winds for runways : did the Author think that a usability might be adopted for temperatures, and, if so, what did he think that it should be ?

The Author had suggested that most pilots would prefer to have over-runs made stronger, rather than verges. In Mr Briscoe's view, the choice would depend on a number of factors, such as the width of the runway and how many runways had been provided, from the point of view of whether it would be necessary to land often with a strong cross-wind. He would also suggest that over-runs were normally required only when there had been an engine failure at the critical point, but engine failures, if they occurred at all, were liable to occur at any time during a take-off rather than at the critical point, and when earlier, the aircraft often ran off the side of the runway. On such occasions it was preferable for the money to have been put into strengthening the verges rather than the over-run.

The Chairman agreed with the Author that runway width was not a function of the basic runway lengths to which the different airport classes referred, and that important changes in runway width had no justification solely on that basis. He also agreed with the Author that where standards were adopted they should be on some scientific basis ; so many standards seemed to have very little scientific basis. The Chairman knew of one busy international airport where the runway was only 100 feet wide, and nobody had ever complained ; the only reason for that was that no one had ever been told. That aerodrome had carried up to 100 aircraft a day, of all sizes.

The Author, in reply, thanked all the speakers for the kind, if at times slightly mixed, reception which they had accorded to the Paper. He had expected a mixed reception, because certain parts of the Paper had been put in deliberately as kites which he had thought might be shot at.

Despite an apparent disparity of view, he wondered if Mr Skinner and Mr Lloyd-Jones were not really expressing the same objective as he had

attempted to formulate in the Paper, when discussing runway lengths. Mr Skinner considered that the I.C.A.O. should specify the safety factors to be allowed in addition to the actual take-off and landing lengths, whilst Mr Lloyd-Jones had said that it was necessary to know what margin to allow over and above the minimum requirement to enable aircraft to operate with an acceptable degree of safety. Although perhaps he had not made it very clear, that was what he had had in mind when he suggested the formulae shown on pp. 288 and 289 of the Paper; it would be for the I.C.A.O. to determine the values of the multipliers X, Y, and Z, so that the resultant runway lengths would provide the safety factor necessary for practical operation. He agreed, however, that such formulae would give only a minimum basic length, and, as Mr Lloyd-Jones had rightly pointed out, it would need to be subject to upward adjustment for adverse local conditions. However, he could not see eye to eye with Mr Lloyd-Jones in saying that the reliability of the aircraft and of their operators should be taken into account; surely that would make the problem immensely complicated and indeed insoluble in the case of international services. It seemed to be necessary to assume a practical and reasonably uniform and efficient standard of operation and maintenance of all aircraft allowed to fly. In the same connexion, he was grateful to Mr Fitt for noting that the European-Mediterranean Regional Group of I.C.A.O. had voiced the need for the publication of more aircraft performance data; that seemed to be a step in the right direction.

The feeling of the meeting about specifying a ratio of the strength of over-runs (now called "stopways") and verges to that of the runways appeared to be mixed: perhaps it was for a similar reason that the I.C.A.O. had "tended to stall" on the question, as Mr Hargreaves had aptly put it. Replying to Mr Lloyd-Jones and Mr Lewis-Dale on that point, the Author felt that an endeavour should be made to assess a fixed "target" ratio of the bearing capacity of a stopway or verge to that of the runway proper, and that that should be recommended to the engineer as something desirable but to be achieved only if it could be done economically. That would leave the question less wide open than the I.C.A.O. had done to date, and the ordinary civil engineer with little knowledge of what would and what would not safely support an aircraft in emergency conditions without hazard—and indeed of exactly what was meant by "hazard" in that connexion—would not be completely in the dark. It was fairly common to reduce the strength of the less frequently used edges of a runway below that of the more heavily trafficked centre portion, and the Author wondered if something could not be arrived at on those lines, using estimates and possibly, later on, statistics of the number of times that the stopways and verges needed to be used. He agreed with Mr Lewis-Dale about the difficulty in the case of grass verges, but that did not apply in many parts of the world. There were aerodrome sites abroad where turf, as it was known in Great Britain, could not be grown and where local

materials already on the site could be used for the construction of stopways and verges but it was essential to import the runway materials. If some desirable ratio of strength were specified, the engineer would find it easier to say whether the economy provided by stopways and verges should receive consideration.

He was glad that Mr Lloyd-Jones supported his view that runway widths required careful consideration. Mr Hargreaves was not in favour of increasing the number of standard widths, and neither was the Author, except that economy had to be sought. What he really objected to were the big increases of 50 feet in single jumps. He appreciated the psychological advantage to a pilot landing a plane on the lowest possible number of standard widths (he had in fact said so in the Paper), but it seemed to be an inordinately expensive way of providing perspective and scale, and one which was no advantage at all when landing conditions were most difficult, by night or in bad visibility. Nor did he believe that the answer lay in standardizing on a 150-foot width of runway only : for example, in certain conditions of high cross-winds on a single runway it might be that widening the runway would be a cheaper practical solution than building a second runway. The Chairman's remarks encouraged him in his hope that there might be further discussion of the question of runway widths.

Mr Lloyd-Jones had queried his remarks about the provision of lighting at Dar es Salaam ; it was not a question of which direction of landing was the more important from the point of view of safety, because obviously both were equal, but of what should be provided in the way of fixed installations in the first instance during the early development of the aerodrome, with limited finances and limited night traffic. In the case of the taxiway lighting, the economy meant only a reduced convenience of taxying route for the less frequently used direction of landing.

He did not quite see the force of Mr Briscoe's argument that jet aircraft were not always penalized when they were unable to take off at their full weight. He would have thought that when dealing with an international airport one would usually want to provide for all possible uses of the runway, not for short stages alone. Nor did he fully appreciate Mr Briscoe's point about usability for temperatures ; was that not already taken care of by I.C.A.O. in their definition of the temperatures to be used in the calculation of the runway length correction factor ? However, he inclined to agree with Mr Briscoe's dislike of the frequent use of the word " standards " in the Paper ; unfortunately, that was inherent in the title, but he had tried to make it clear that they were not rigid in most cases and that reference to and consultation with the aeronautical side was essential in their application. It was often the case abroad that the engineer had to stand very largely on his own in any matter of an airport and that was the main reason for the Author's plea that things should be made easier for him, and for his suggestions for doing so.

The other points made in the discussion seemed to have been answered

by the recommendations of the Fifth Session of the AGA Division of I.C.A.O., which had been mentioned by Mr Hargreaves and whose Report was now publicly available. It was gratifying that so much progress had been made at that Session.

Correspondence

Mr Robert Marwick, of Nairobi, observed that the presentation of the Paper was timely, since many Authorities were faced with the task of fulfilling the requirements which were arising through the introduction of jet airliners on trunk routes and of larger aircraft on regional services. The financing of the necessary work was becoming increasingly difficult and airport designers were thus under a strong obligation to reconsider the fundamentals of their work in relation to the trends of aeronautical requirements and practices. Although I.C.A.O. had not imposed on aircraft designers any maximum standards for such requirements as runway length and bearing capacity, it appeared that restrictions imposed by the size and strength of existing airports had been having an equivalent effect in recent years. That trend, coupled with the successful development of turbine engines, had enabled the envisagement of some maximum airport standards, including runway lengths for jet airliners, which were well within the provisions of Annex 14 to the Convention on International Civil Aviation. He understood that, in Britain, the length requirements of the Comet 1 were being regarded as maxima and he had recently noted that the United States Air Transport Association had specified similar maximum runway basic lengths—5,500 feet for landing and 7,500 feet for take-off, for a 70-80 seater jet airliner of 3,200 miles range ; the latter specification also required that the aircraft should be capable of landing in a 40-mile-per-hour cross-wind. The consideration of such trends, together with those of operating practice, might well affect the manner in which the currently required improvement, or replacement, of an airport was carried out.

After the question of the number and orientation of the runways, which the Author had covered very fully, the possibility of reducing the fully paved length of runway offered the greatest scope for possible economies, since it affected the length of any parallel taxiway joining the traffic apron to the runway ends ; the possible savings included reduced costs of providing lighting and drainage for those two pavements. It was not yet provided for in Annex 14 ; the operational Standards laid down in Annex 6 did, however, appear to cover its use, since they allowed the inclusion of any available additional length between the runway end and the commencement of the clearance plane in the climb zone to be included in the distance required to be available for take-off and climb to 50 feet after failure of the initial engine ; they also permitted the completion of accelerate-stop movements on " suitable ground " beyond the runway.

Fig. 2 showed that the total length of a shortened runway plus one over-

run equalled the length of a normal runway. A further economy was, however, permissible where, owing to temperature and altitude conditions, the length required by the critical aircraft to take-off to 50 feet after an engine failure exceeded both its accelerate-stop distance and its required landing runway length ; the strip length would be determined by the take-off distance but part of it need not be paved. For take-off, the ratio between the runway plus over-run length and the strip length might, in theory, be varied from that determined by the two normal performance distances ; that would, however, vary the critical speed of a particular aircraft and thus possibly increase the risk of a pilot making an error of judgement between continuing or stopping a take-off in the event of engine failure ; and the required landing runway length might restrict the extent to which the runway plus over-run length could be shortened in that way.

It had once been common practice to make the landing runway length $\frac{100}{70}$ times the landing distance for I.F.R. conditions or $\frac{100}{80}$ for V.F.R. conditions and, owing to the comparatively small effect of temperature and altitude on landing distances, the landing runway length thus obtained was often appreciably shorter than that for take-off. Annex 6 to the Convention, however, now required that the landing distance for the critical aircraft should not exceed 60 per cent of the available length and the longer landing runway length thus required might, in some circumstances, exceed the take-off length. The length of high-quality surfacing could, nevertheless, still be reduced, since the more extreme landings, such as occurred in bad visibility conditions, could be completed on an over-run ; but it might not be desirable to reduce it to the landing distance only, since that might give pilots an impression of inadequacy. The landing runway length beyond the landing distance also required a higher-quality pavement than an accelerate-stop over-run, because abnormal landings were more frequent than uncompleted take-off movements ; but it did not have to be uniform, since the frequency of abnormal landing movements decreased with distance. The appropriate solution in the case of a runway to be used in one direction thus appeared to be the provision of full pavement over a little more than the normal landing distance ; then surfacing of the same appearance, but not necessarily of the same thickness, on a tapering base-course for the remainder of, say, 80 per cent of the landing runway length ; and, finally, an over-run.

It thus appeared that the runway, over-run, and strip lengths should each be determined from separate basic lengths and temperature-altitude correction factors, rather than by fixed proportions of the actual length of the equivalent normal runway ; otherwise one type of movement might not be fully provided for.

The more extreme reductions might not be economical when a runway had to be used in both directions and restrictions arose where, in addition, an instrument landing system was, or might be, necessary. The localizer

beacon could be set back beyond the over-run, but the inner marker would have to be at a standard distance from the threshold and the runway itself would therefore have to have at least the required landing runway length. The beacon could not be sunk in the accelerate-stop over-run but the placing of the aerials only on the extended centre-line of the runway, with the transmitter placed 200 feet to one side (as recommended in Annex 10), might be accepted as a risk in consideration of the low frequency of accelerate-stop movements.

The strip in the proposed lay-out for the new airport at Nairobi extended at least 1,000 feet beyond the ends of the runway ; further reductions might be made but the more extreme measures, which might be theoretically possible, might not be practical or economically effective. Similar proposals were under consideration for increasing the effective lengths on the new airport which was under construction at Dar es Salaam.

Mr Marwick pointed out that the Load Classification system in Annex 14 gave no more guidance to the designer than did the parallel system for runway lengths. Studies in recent years had shown the earlier estimates of equivalent wheel loads to be excessive for most pavements and I.C.A.O. had recently given more realistic figures for particular aircraft ; it should, however, be borne in mind that the equivalent wheel load was a function of the depth below the surface of the critical layer of the subgrade.

The Author had mentioned the need for guidance on the bearing capacity of over-runs. Mr Marwick suggested that that could be obtained from the chart in the United States Army Engineers Manual showing the percentages of the full design thickness which should be adequate for wheel coverages down to twenty in number ; that could be also used to design part of the runway to reduced standards for lower frequency of operations. Further guidance might be obtained from shear-strength methods of pavement design by reducing the factors of safety which they included.

The Author, in reply, welcomed Mr Marwick's submissions : he considered that his discussion of the factors affecting the achievement of economy of paved runway length would materially assist towards an understanding of the principles now made public by I.C.A.O. in Document 7341, the Report of the Fifth Session of the AGA Division, to which he had already referred in his reply to the oral discussion.

He agreed that an acceptable solution to the problem of the inner marker might be found along the lines which Mr Marwick suggested, the aerial only being placed within the stopway or " over-run " and being made frangible ; it did seem, however, that 300 feet might be more acceptable to the interested authorities as the distance by which the transmitter would be placed to one side of the centre-line. The intention of the increased distance would be to provide a margin for wing-tip clearance.

Correspondence on the foregoing Paper is now closed and no further contributions will be accepted —Sec. I.C.E.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

6 January, 1953

Lieutenant-Colonel R. H. Edwards, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Maritime Paper No. 22

"The Reconstruction of Greenwell's No. 1 Dry Dock and Ancillary Works at Sunderland"

by

Harry Ridehalgh, M.I.C.E.

SYNOPSIS

The Paper deals principally with the construction of the new No. 1 Dry Dock at Sunderland, on a site previously occupied by a smaller dry dock, built in about 1860.

The difficulties encountered before the main cofferdam could be built are described; these were attributable in part to the proximity of the new entrance to the entrance to the main dock system of the port, and also to the presence of badly fissured substrata.

Advantage was taken of the high rock levels in the body of the dock in preparing the design of the barrel walls, and by venting the floor it was possible to keep it relatively thin.

The new dock was connected by tunnel to the pumping system of the adjacent No. 2 Dry Dock, and by installing only one additional pump it was possible to use the No. 2 Dock pumping equipment for both docks.

The largest "Box" flap gate so far constructed is used to close the entrance to the new dock. The most interesting feature in this connexion is the use of steel-to-steel meeting faces, in which a rubber insert ensures the necessary water seal.

While the reconstruction of No. 1 Dry Dock was proceeding, No. 2 Dry Dock was also extended by 50 feet and the existing fitting-out quay was lengthened by 210 feet. Both these extensions were carried out while the existing facilities remained in use.

Continuous gravity fenders were provided along the face of the fitting-out quay, which is of reinforced-concrete construction on vertical and raking piles.

Introduction

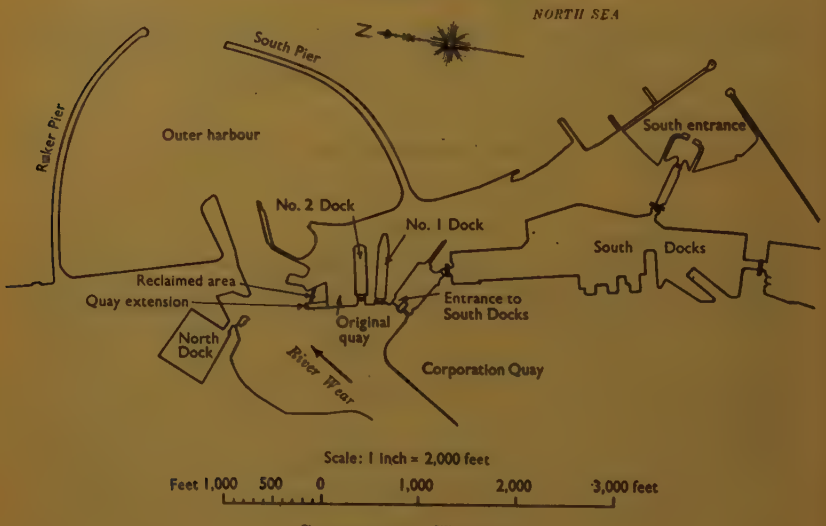
Messrs T. W. Greenwell & Co. Ltd carry on their business of ship repairing at their yard adjacent to the entrance to the South Dock at Sunderland, and prior to the outbreak of the 1939–45 War they used their Nos 1 and 2 Dry Docks and a short fitting-out quay principally for the purpose. Normally, the minimum of repairs are dealt with in the dry docks, and vessels are afterwards passed to the fitting-out berth for completion afloat.

The position of the area under consideration in the port can be seen by

reference to *Fig. 1*, and the relative positions of the dry docks and fitting-out quay will be noticed. From the Figure it will be appreciated that the length of fitting-out quay originally available was insufficient to ensure that a vessel lying alongside it would, under all conditions, be clear of the entrances to the dry docks.

During the war, No. 1 Dry Dock sustained major damage to the entrance and the pumping facilities, and was rendered inoperative. This, together with the inconvenience of docking vessels when the fitting-out quay was occupied, and the growing demand for the repair of larger vessels, led to the reconstruction programme which is described in this Paper.

Fig. 1



SUNDERLAND HARBOUR

The Company retained the late Mr N. G. Gedye, O.B.E., B.Sc., M.I.C.E., to prepare proposals for constructing a new and larger dry dock on the site of the old No. 1 Dock and for extending the fitting-out quay, at the same time reclaiming approximately an acre of foreshore behind the quay and adjacent to their workshops. These designs were completed shortly before Mr Gedye died in 1947 and Sir William Halcrow & Partners were thereupon called in to complete the work.

Various circumstances subsequently combined to show that it would be of advantage to prepare new designs and the work was eventually carried out in accordance with the details here described.

The work was commenced by the civil engineering contractors, The Demolition and Construction Co. Ltd, in May 1950 and was completed in the form shown in *Fig. 2* during September 1952.

Aerial View of Docks

Fig. 5

No. 1 Dock looking West from Head of Dock

Fig. 7

Excavation for North Wall of No. 1 Dock

Fig. 10

BEARING FOOT ON GATE

Fig. 11

SOUTH BEARING FOR NO. 1 DOCK GATE

Fig. 14

Fitting-Out Quay Extension
(Part View)

Site Information

The spring-tide range in the port of Sunderland has recently been redetermined as being 14 feet 9 inches and datum for the purposes of the work was the River Wear Commissioners' old datum, which is 1 foot 4 inches below mean high water, spring tides. A swell of about 2 feet in range was experienced during the period of construction in the area of the harbour in which the work was carried out, but this is known to have been considerably exceeded at other times.

Indications of the type of substrata likely to be encountered were forthcoming from preliminary reports made by Mr Gedye and these were later amplified by boreholes and trial pits sunk in appropriate positions throughout the site. These showed that made ground, sand, and gravel would be encountered down to a level of — 8·00, where rock was recorded.

The " rock " was later found to be made up of alternating thin layers of dense sandy marl and beds of soft and medium-soft Magnesian Limestone traversed by solution channels. The sandy marl deteriorated in places to thin gravel beds, and the soft limestone was interspersed with pockets of very hard limestone in the form of large deposits of crystalline spheres, some of which measured as much as 1 foot in diameter.

The whole of the substrata was traversed with water-bearing fissures and the beds rose from the dock in the direction of the main shipping channel at the entrance to the South Docks.

Local information also revealed that some difficulty was likely to be experienced in obtaining an effective seal when using steel-piled cofferdams toed into such rock.

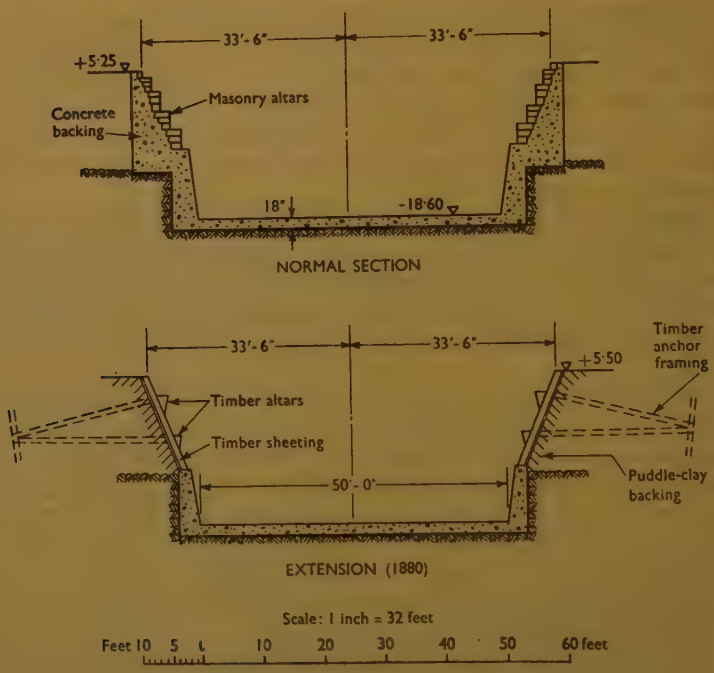
The site was severely restricted, as often is the case on works of reconstruction. Little space was available to the contractor for working areas, access was poor, parts of the site were occupied by buildings scheduled for demolition as soon as they could be replaced, and the new dry-dock entrance lay immediately adjacent to the entrance to the main wet-dock system of the port which had to be kept open to shipping.

No. 1 Dry Dock

Original Dock

The old dock, which was demolished in the present scheme, had been built in about 1860 and widened and extended in 1880. It was about 430 feet long and had a sill depth of 20 feet at high water, spring tides, with an entrance width of about 50 feet. Chain-operated timber mitre gates closed the entrance where the principal war damage had been sustained. Serious damage was also sustained by the pump-house and pumping equipment, but these would in any case have been quite unsuitable for incorporation in the new work. Cross-sections of the old dock are shown in *Figs 3* and indicate the profiles adopted towards the end of the nineteenth century.

Figs 3



TYPICAL SECTIONS OF OLD NO. 1 DOCK

Description of Work

The new dry dock was designed so that tankers of up to 32,000 tons dead weight could be admitted. It is 675 feet long from gate meeting-face to the head wall, has a clear entrance-width of 87 feet 6 inches, and a depth of 27 feet 4 inches of water over the sill at M.H.W.S.

The dock is so orientated that vessels when docking have fair clearance off the end of the Corporation Quay, which means that the new and old dock centre-lines are not coincidental. Nevertheless, the new entrance lies almost astride the old entrance and the new dock can be said to have been built round the old one. Whilst this was of some advantage, in that a substantial amount of excavation was eliminated, the contractor was denied other advantages which would have resulted from having a dumpling to work from.

The original pumping facilities lay within the area occupied by the body of the new dock, and the opportunity was taken to centralize all pumping facilities for both Nos 1 and 2 Docks in the adjacent underground pump-house of No. 2 Dock, which had been built in 1925 and was equipped with electrically driven pumps. This was achieved by constructing a new drainage culvert in tunnel between the new No. 1 Dock and the existing

No. 2 Dock pump-house sump. By redisposing the pipe connexions within the pump-house, and introducing an additional 10-inch-diameter centrifugal drainage pump and new sluices, the original pumping equipment was made to serve both No. 1 and No. 2 Docks. Direct flooding of the new dock was made possible by the provision of a new 54-inch-diameter culvert between the north entrance wall and the north barrel wall. Dock-side services were provided in special ducts and recesses, and included 8-inch-diameter salt-water, 6-inch-diameter fresh-water, 6-inch-diameter compressed-air, 6- and 8-inch-diameter oil, and 4-inch-diameter steam mains, in addition to electric lighting, power, and welding points accessible from the top altar.

A new 23-foot-gauge track to carry a 40-ton-capacity travelling crane was constructed along the north side of the dock, the front rail being carried on the top of the dock wall and the back rail on longitudinal beams supported on piles. Foundations on which a similar crane could be erected were constructed on the south side of the dock.

A general plan and sections showing the general form of No. 1 Dry Dock as constructed are shown in Figs 4, Plate 1, and *Fig. 5* is a view of the dock shortly before the cofferdam was removed.

Programme

At the outset it was evident that the contractor would not be able to prepare a comprehensive programme for the whole of the work, for the release of the eastern end of the site was dependent upon the provision by the River Wear Commissioners of new accommodation to replace that afforded by the buildings on the site scheduled for demolition. Inclement weather and delays in delivery of materials seriously affected the Commissioners' building schedule, as a result of which the programmes of work prepared by the dock contractor had to be revised on at least four occasions. Close co-operation between the parties did, however, ensure that work on the dock was never brought to a standstill, but there is little doubt that the enforced short-term planning led to the uneconomical disposition of plant and equipment.

A further restriction on progress resulted from the need for keeping the entrance to the South Docks open to the general shipping of the port. As a result, it was not possible to build, at the outset, a cofferdam which would embrace the whole of the new entrance. The new south roundhead had first to be built inside a subsidiary cofferdam and then be used as a landing for the southern end of the main cofferdam.

The construction programme began with the diversion of existing services clear of the new site, and excavation in open cut and trench then proceeded in an easterly direction from a point just behind the old entrance. Excavation was held up for 3 months when it had reached the area round the head of the old dock, but the eventual release of the whole site enabled the contractor to demolish the buildings, carry out

construction of the new head and bow walls in trench, and join up with work completed at the western end of the site.

In the meantime the construction of the new south roundhead and the main cofferdam had been completed, after which construction of the outstanding work was pushed ahead rapidly to conclusion.

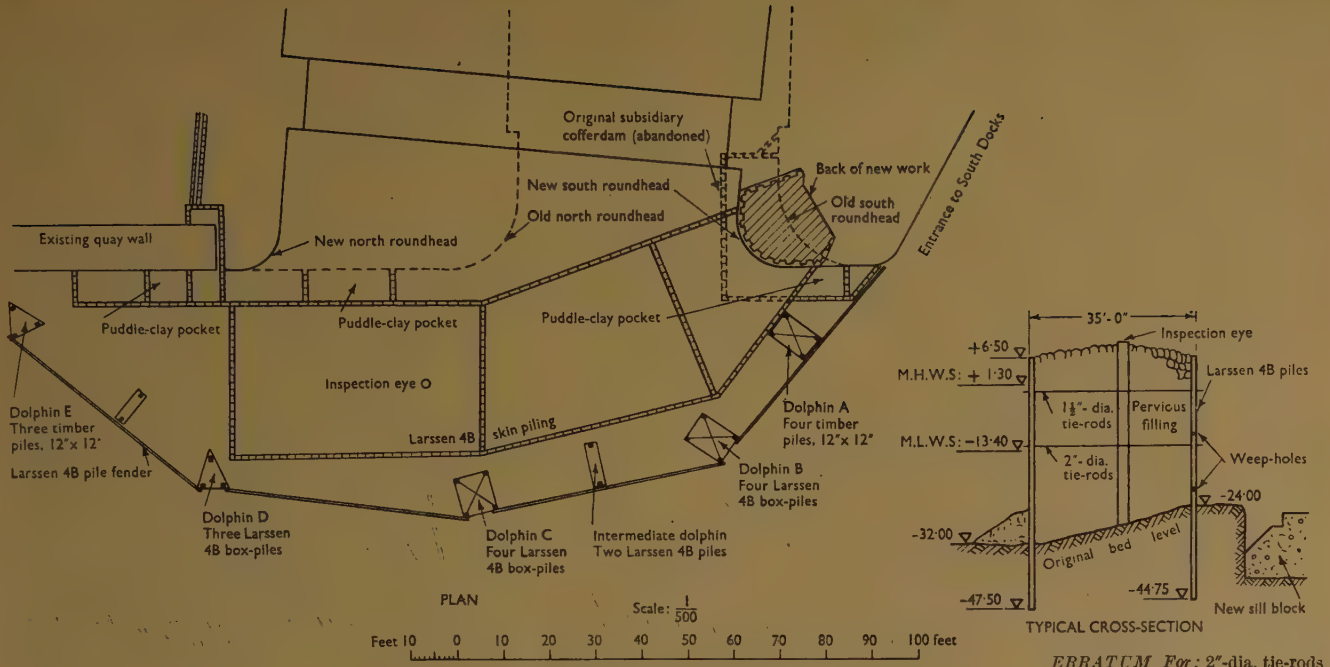
It is not improbable that, had commencement of the work been delayed for about 6 months, the contractor would, in the light of subsequent events, have found it possible to reduce the construction period by a similar amount, probably with financial advantage to the owner.

Cofferdams

As explained previously, it was necessary to complete the new south roundhead before construction of the main cofferdam could proceed, and for this purpose the contractor built a small single-skin subsidiary steel-pile cofferdam round the area. The piles were driven about 5 feet into the limestone. In places this necessitated first cutting through the concrete apron of the old entrance by using a 12-inch-by-8-inch rolled steel joist shod with a rock-chisel. Demolition of the old concrete and masonry roundhead proceeded inside the cofferdam as the water level was lowered and strutting was introduced. As two 5-inch-diameter pumps developed a head of about 20 feet of water inside the dam, blows developed under the toes of the piles and it became evident that pile penetration was inadequate. Subsequent attempts at re-driving merely produced a rigid iron-bound condition in the piling, so that it became clear that other means would have to be adopted for constructing this roundhead. Before the cofferdam was abandoned, however, the footings of the lead-in wall to the South Dock were exposed and successfully underpinned in concrete. Since the loss of time occasioned by this failure showed signs of interfering with the main cofferdam programme it was decided to revise completely the design of the roundhead so that it could be constructed in the wet without the need for providing a watertight cofferdam. Permanent steel sheet-piling was therefore driven along the actual face line of the new roundhead and turned back into the body of the old work along lines which it had been possible to clear during the life of the original cofferdam. The foundation level of the new roundhead was raised in view of the introduction of the permanent piling, and work then proceeded under water on the cutting away of the remaining portions of the old roundhead and the removal of the underlying rock down to foundation level; the area enclosed by the sheet-piling was subsequently concreted without any effort being made to pump dry. Piles driven to level — 49·00, giving an average penetration of about 20 feet below original bed level, were cut off at + 5·00 and the roundhead was finally completed by placing a 3-foot-thick cap of concrete across the head, the whole being tied back to existing work.

Completion of the south roundhead enabled work to proceed on the construction of the main cofferdam, which was of the double-skin porous

Figs 6



ERRATUM For: 2"-dia. tie-rods
read: 3"-dia. tie-rods

TEMPORARY COFFERDAM

fill type as illustrated in *Figs 6*. An effective vertical seal was obtained at the southern extremity because the two sheet-pile skins were keyed on to piles forming part of the roundhead, but at the northern end, sheet-piling had to be returned some distance shorewards through a 10-inch gap which fortunately existed in the original quay wall, before the ingress of water was finally cut off. The puddle-clay pockets shown in *Figs 6* are supplementary to, and not necessarily essential parts of, the design. In view of the experience gained with the original small cofferdam for the south roundhead, the contractor was instructed to drive all piles to a toe level of at least — 47·50. It appeared that sandy strata and faults in the limestone on the site of the cofferdam rose to bed level of the harbour some distance off-shore, and it was in an effort to cut these faults that the piles were driven to the level stated.

It may be added that extremely hard driving resulted from these instructions, and on withdrawal it was found that several of the pile toes had been quite badly damaged. This trouble may in part be attributed to the initial use of a No. 10 McKiernan-Terry hammer. Subsequently this was replaced by a No. 7 hammer with slower driving but better results. Nevertheless, the wisdom of the decision was proved, for the volume of water passing the cofferdam eventually dropped to about 0·5 cusecs, though it may perhaps be adjudged fortunate that the positions of the damaged piles did not coincide with water-bearing fissures.

Withdrawal of the cofferdam was commenced after the new entrance was completed, the dock being first flooded through No. 2 Dock flooding system. Piles were withdrawn by Zenith extractors without undue difficulty and the hearting was removed by the River Wear Commissioners' bucket dredger.

Dock Construction in General

In the early stages of the work the contractor found it possible to close the old damaged timber mitre gates and to pump the original dock dry. By supporting the gates with temporary mass-concrete buttresses built on the old sill he maintained them in service for a considerable period and so made it possible to carry out a substantial amount of work on the body of the new dock prior to the completion of the main cofferdam.

Soft excavation was in general carried out down to rock level in open cut by an R.B. 37 excavator fitted with a 1½-cubic-yard dragline bucket working eastwards from the back of the old entrance. The rock could be broken out by pneumatic tools and stood to a good face, so that rock excavation for the barrel walls was in untimbered trenches as shown in *Fig. 7*. The head- and bow-wall trenches were, however, carried down as such from ground level and in consequence were sheeted with steel piles and timbered in the usual way.

Excavated material was used initially to form an access ramp down to the old dock bottom, up which spoil from all subsequent dock demo-

lition was removed, by lorry, to the reclamation site behind the new fitting-out quay or to the River Wear Commissioners' spoil site at Hendon.

It was found that by using small explosive charges the limestone could be won by R.B. 19 shovels or skimmers, and this method was used for excavating the rock down to the underside of the new floor.

The dock walls, provided with 2-foot-wide cantilever altars at + 4·00, — 6·00, and — 14·00, were backed with rubble drains led to pressure relief valves at the base. The walls were designed to retain damp fill above rock level, water standing up to 10 feet above floor level, a ground surcharge of 5 cwt per square foot, and point loads from the dockside cranes.

Typical cross-sections can be seen in Figs 4, Plate 1. The walls were concreted in lifts of about 3 feet 6 inches and in blocks 50 feet long, the joints between blocks being sealed with a mixture of tar and bitumen retained in precast concrete units as illustrated also in Figs 4. The reinforced-concrete cantilever altars were cast in situ as the walls were brought up, but the 3-foot-wide ramps giving access to the floor were cast in situ after the walls had been otherwise completed.

The dock floor was 5 feet thick at the centre, 4 feet 6 inches thick at the walls, and was laid with a longitudinal gradient of 1 in 400 to the main drainage sump near the dock entrance. A system of rubble drains was laid under the floor and, as in the case of the wall drains, these drains led to the pressure relief valves at the base of the walls.

Concreting of the floor was carried out in transverse strips 12 feet wide, adjacent strips being divided into two and three blocks in plan in order to break joint. All blocks were concreted up in one continuous operation. The strips were stopped off at between 7 and 10 feet from the walls and the spaces thus formed down each side of the dock were used as temporary longitudinal drains into which all ground-water was turned. A plan of the floor blocks is shown in Figs 4, Plate 1. These temporary longitudinal drains were concreted up as one of the final operations in the floor-construction programme.

Permanent 24-inch-by-9-inch open drainage channels down each side of the dock collected the water from the pressure relief valves and delivered it to a small sump inside the main drainage culvert from where it was finally cleared by a 10-inch-diameter centrifugal pump.

For wall and floor concrete, 2½-inch gravel from the upper reaches of the River Wear was used and was invariably washed on the site before use. The supply of suitable sand was a difficult problem and best results were obtained when a West Hartlepools sea sand was used with the river gravel.

Proportions adopted were :

13·5 cubic feet of 2½-inch river gravel
6·8 cubic feet of West Hartlepools sand
2 bags of Portland cement.

This mixture gave 15·3 cubic feet of concrete in place.

The specified strength of 2,550 lb. per square inch at 28 days with a water/cement ratio of 0·58 was, however, difficult to attain with the materials available in the district.

The relatively small quantities of concrete to be placed in one operation and the widely spaced concreting areas made central mixing out of the question. Combinations of ½-cubic-yard, ¾-cubic-yard, and 1-cubic-yard mixers were disposed around the site and finally on the dock floor from where R.B. 10 and R.B. 19 cranes, and 7-ton derrick cranes were fed by jubilee wagons.

While the work of building the body of the dock behind the new entrance was proceeding, a shaft was sunk on the line of the new drainage culvert between No. 1 Dock and No. 2 Dock sump, and from this shaft the culvert was excavated in both directions in tunnel. The shaft was eventually lined in concrete and formed the chamber for the main sluice between No. 1 Dock and the pump-house.

The culvert was 6 feet by 6 feet in section and is shown in Figs 9, Plate 2, together with details of the sluice and operating equipment. It was the original intention to use hydraulically operated sluice valves at this and all other culvert control points, but it was found impossible to obtain such equipment in the time available.

Entrance and Gate

On completion of the main cofferdam, work was commenced on the new entrance. The old gates and sill were removed, roundheads demolished, and the new work previously completed on the body of the dock was extended westward to the new entrance. The old north roundhead had been founded partly on a steel caisson filled with concrete, through which the line of the new roundhead passed. Both the old concrete and the old steelwork were found to be in good condition and it was possible to dress back the old concrete to form a good face for the underwater portion of the new roundhead. The upper part of the new north roundhead and the northern gate rest were founded on the old caisson.

The entrance sill, which was 14 feet thick, was cast in four blocks each of two lifts, vertical joints being sealed with the form of bitumen joint illustrated in Figs 4, Plate 1.

The entrance to the dock is closed by a welded steel Box-type falling leaf-gate spanning 91 feet 2 inches between centres of bearing faces, and is designed to withstand 32 feet head of water under maximum tidal conditions.

The original design of the entrance meeting faces was on orthodox lines, with vertical seals made by greenheart timbers on the gate, which would bear on dressed granite quoins, and the horizontal seal along the sill consisting of a circular greenheart keel on the gate, rotating in a continuous cast-iron bearing built into the concrete sill.

It was found, however, that economies could be effected by adopting a

type of meeting face evolved by Sir William Arrol & Co. Ltd, for two similar but smaller gates previously installed at Falmouth, which, in addition, incorporated an interesting departure from usual practice so far as the horizontal bearings were concerned.

The meeting faces, both horizontal and vertical, consist of an 8¾-inch-by-3⅜-inch mild-steel billet welded to the gate perimeter which, with the gate in the closed position, finds a bearing against a 4½-inch-by-¾-inch stainless-steel plate cast into the concrete of the entrance. The steel billet is recessed in its bearing face and carries a projecting replaceable rubber strip insert, which is continuous round the whole meeting face and ensures a watertight seal.

By adopting this type of meeting face, the need for the continuous horizontal circular keel has been eliminated, and the gate is instead provided at each end of its base with massive mild-steel bearing feet which, when the gate is raised or lowered, rotate in mild-steel bearing shoes built into the entrance walls at the junction with the sill. Details of the meeting faces and bearings are shown in Figs 8, Plate 1, and in *Figs 10* and *11*.

By this departure from standard design all expensive granite construction for the entrance was eliminated, together with all the problems of accurately setting the granite quoins and the precise dressing of the meeting faces. Furthermore, it became possible to abandon the use of greenheart timber completely, which in turn eliminated one of the frequent sources of leakage at dry-dock entrances.

The financial saving effected by the change-over in gate design was approximately 5 per cent of the cost of providing a gate with greenheart meeting faces and continuous keel, plus the additional saving achieved by eliminating about 350 cubic feet of granite quoin costing roughly 100*s*. per cubic foot in place.

To accommodate the steel meeting-face assembly, which had to be cast into the concrete, a continuous recess measuring 2 feet 6 inches by 1 foot 7 inches was formed all round the entrance, into which ⅞-inch-diameter setting-up bolts projected. The meeting-face assembly (shown in Figs 8, Plate 1) consisted of a welded stainless-steel T, the head being 4½ inches by ¾ inch and the leg, 12 inches by ¾ inch. Brackets and angles were welded on to the leg which coincided with the setting-up bolts cast into the recess. The assembly was made up generally in 18-foot lengths and welded into a continuous strip round the entrance after setting in position. Mild-steel reinforcement secured to the leg of the T was provided in appropriate places to transmit the load from the assembly into the body of the entrance walls.

The steel meeting face was first held loosely in position on the setting-up bolts and then set securely and accurately by the locking nuts relative to piano wire strung and tensioned across the entrance. Continuous checks by theodolite ensured absolute precision.

The accurate setting of the bearing shoes was of paramount importance,

for not only was it essential to ensure that the individual axes were truly horizontal and vertical but the greatest care had to be taken to ensure that the two pivot points lay on the same horizontal axis. By the use of piano wire, levels, and theodolites, and a process of successive approximation (each shoe weighs about 8 cwt), absolute precision was again achieved.

It might here be remarked that, as a consequence of the new design, the work of setting the meeting faces and bearing shoes was transferred from tradesmen to engineers, with undoubted advantage.

The dock gate was built by Messrs Vickers-Armstrongs Ltd, in their Naval Yard at Walker-on-Tyne, and was then launched and towed to the site at Sunderland. It is of all-welded, mild-steel construction, spanning 91 feet 2 inches between centres of vertical meeting faces and having a height from horizontal seal to top deck of 31 feet 10¼ inches, and a moulded thickness of 6 feet 6 inches.

Seven horizontal decks at 4-foot centres are provided and the length is divided by nine vertical bulkheads at about 9-foot centres. Of the internal compartments so formed, eight are used as buoyancy chambers, these being located between decks Nos 4 and 6. All chambers are accessible through oval manholes, generally 20 inches by 14 inches in size, and those above the fourth deck are provided with grated openings on the seaward face to permit the entry of temporary water-ballast. The lower chambers are watertight and are provided with flooding valves for the admission of permanent water-ballast.

Skin plating is 0·62 inch thick, with 0·70-inch-thick doubling plates over the decks. Plating to the air chambers is 0·50 inch thick and all other deck and bulkhead plating is 0·44 inch thick. Stiffeners out of 3-inch-by-3-inch-by-½-inch angles are used in all panels. The total weight of steel in the gate is approximately 175 tons.

The steel billet forming the meeting face is continuous round the sides and bottom of the gate but at each bottom corner it is replaced by a short length of 4⅛-inch-thick billet out of which the bearing feet are formed. The lower face of each foot is shaped to a 12-inch radius and seats into the built-in shoe, whose bearing face is lined to the same radius with ½-inch-thick stainless steel.

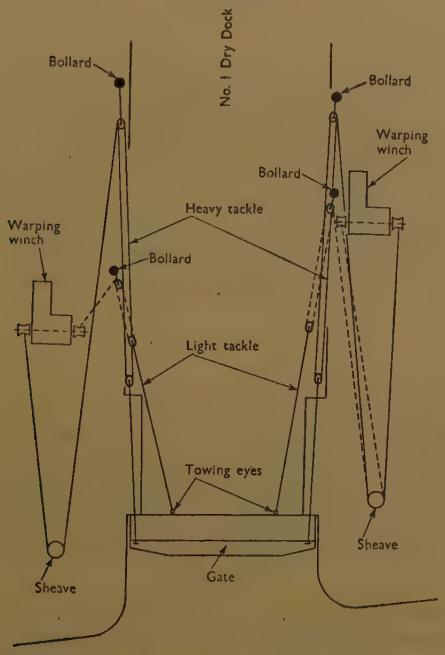
When the first Falmouth gate was docked, after a period of 4 years, an examination revealed that there was slight corrosion on the half-face of the meeting billet which had been constantly submerged and in direct contact, when the gate was closed, with the stainless-steel member built into the entrance wall. This corrosion was approximately 0·004 inch deep and was attributed to the differing electro-chemical properties of mild and stainless steels. As a precaution, therefore, a 2-inch-by-½-inch replaceable zinc strip has, in the present gate, been studded to the side of the mild-steel billet round the gate, and the whole of the embedded meeting-face assembly—instead of simply the head, as originally intended—has been made in stainless steel.

Hinged flaps were provided along the base of the inside face of the gate to prevent debris from becoming lodged between the sill meeting faces.

All steelwork was pickled in a 5-per-cent solution of hydrochloric acid, and after erection all exterior surfaces of the gate and the interior surfaces of the air compartments were given two coats of bituminous solution and one coat of bituminous enamel applied hot. Grease paint was used for the interior surfaces of the water compartments.

The gate is operated by an electrically driven spur-geared winch located

Fig. 12



TACKLE ARRANGEMENT FOR STEPPING GATE

below cope level on the north side of the entrance. A 65-brake-horse-power squirrel-cage motor drives the winch through scoop-controlled fluid couplings, operation being carried out by controls at cope level. The hauling rope is anchored on the south side and a single winch is used owing to space limitations.

The gate can be lifted or lowered in 4 minutes by motor or in 40 minutes by hand-operated gear coupled to the winch.

The general arrangement of the gate is shown in Figs 8, Plate 1.

The arrangements for stepping the gate are shown in *Fig. 12*, the light tackle being used to draw the feet of the gate up to the dock meeting face, and the heavy tackle to raise the gate into position.

The gate was floated into the entrance in an almost horizontal position and, after tackle had been secured, the two flooding valves serving the water-ballast tanks were opened. Air escaped through vents at each end of the gate, intermediate water-ballast-tank bulkheads being provided with air escape holes.

The foot of the gate sank on an even keel until it was drawing about 10 feet of water and then surged, the ends reaching a maximum difference in level of between 3 and 4 feet until, when finally brought on even keel again by varying the water ballast, the gate was drawing roughly 16 to 17 feet. It was then a comparatively simple operation to haul on the tackles and enter the feet into the bearing shoes and bring the gate into the closed position.

Investigations have proceeded as to the causes of the surging reported above, and indications are that the reduction in length of the water-ballast chambers might prevent similar occurrences.

The deflexion of the centre of the top of the gate under a head of 26 feet of water was approximately $\frac{1}{8}$ inch and there was no trace of any water passing the meeting faces.

The cost of the gate together with hauling winch and equipment was approximately £39,000, which represents about £220 per ton of steel in the gate itself.

General

The original pumping installation in No. 2 Dock comprised two 36-inch-diameter centrifugal pumps, one 9-inch-diameter drainage pump, and one 10-inch-diameter ballast pump, and it was necessary to add only an additional 10-inch-diameter centrifugal drainage pump. This equipment can dewater No. 1 Dry Dock in 1 hour 50 minutes when a vessel is docking. The dock can be flooded-up in about $2\frac{1}{2}$ hours when not occupied.

Travelling cranes alongside No. 1 Dock and on the fitting-out quay use a plough-type electric pick-up from an underground conductor trench. When in operation, steel flaps overlying the trench slot are lifted one by one by the plough as the crane travels and the conductors are only exposed for about 5 or 6 feet at the plough. Extensive road and rail diversions were made and new lengths laid round the dock area, and the land between Nos 1 and 2 Docks was graded and surfaced as a working area.

The dock is lit from the cope by floodlights on the top altar, which can be retracted into recesses in the dock walls. Warping winches for docking are provided on each side of the entrance and at the head.

Cost

The cost of reconstructing No. 1 Dry Dock, inclusive of demolishing the old structure and providing a new gate, was about £411,000. Of this, about £43,000 represented the increase in the cost of labour and materials

during the $2\frac{1}{2}$ years the work was proceeding. The value at tender date of the work executed was therefore about £368,000 and, since labourers' hourly pay was then 2*s.* 4*d.*, the dock can be said to have cost 3,160,000 labourer-hours. Craftsmen's hourly pay at that time was 2*s.* 10*d.*, and the approximate proportion of this class of labour employed on the whole of the work was about 23 per cent.

The volumetric unit cost was about 3*s.* 5*d.* per cubic foot measured between walls and from cope to floor.

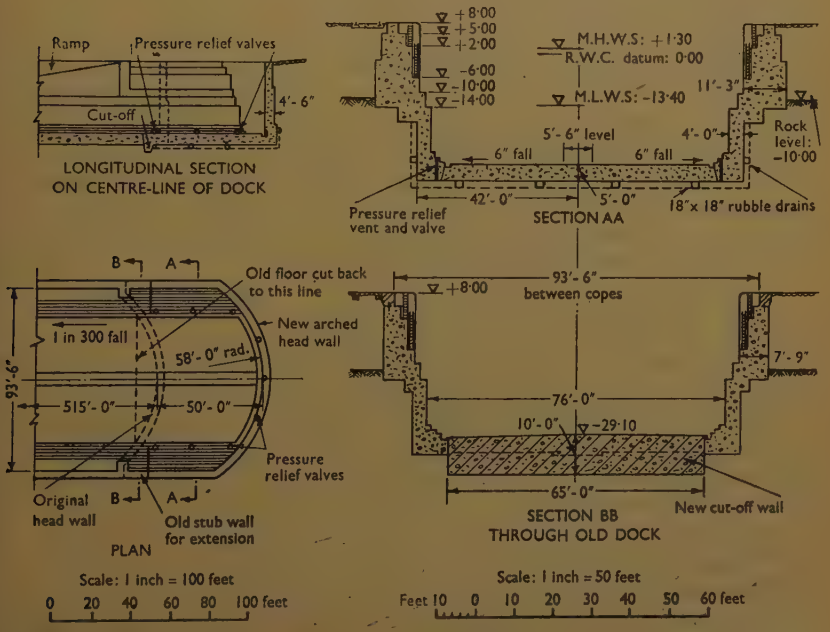
No. 2 Dry Dock Extension

This portion of the work consisted of lengthening No. 2 Dry Dock from 515 to 565 feet, and was greatly facilitated by the dock owners' ability to arrange for a minimum of dockings during the construction period.

It was further facilitated by the fact that, when built in 1925, the barrel walls were continued slightly beyond the head wall in case an extension such as the one now completed should be required at any time.

A general arrangement drawing, showing a plan and sections of the work carried out, is reproduced in *Figs 13.*

Figs 13



Extension of No. 2 Dry Dock

Site Conditions

Soft Magnesian Limestone interspersed with layers of hard " cannon ball " limestone was overlain above level — 12·50 with assorted sands, gravel, and clay. Rock was easily removed by pneumatic tools and the most troublesome material encountered was in the old coast protection works at the head of the dock which had been formed of large boulders of slag laid in concrete.

It was expected that some subsoil water would be encountered during construction in view of the reclaimed ground on which the work was to be carried out.

Programme

The construction of the walls was carried out in four blocks—south wall, south head wall, north head wall, and north wall. Removal of the dumpling in the extension proceeded in step with the concreting, and the demolition of the old head wall followed the dumpling down. Floor concreting was the last operation.

Construction

Excavation for the walls proceeded in trench 22 feet wide, lined with Appleby-Frodingham II steel sheet-piles, and practically all excavation was carried out by hand, loading into tipping skips handled by an R.B. 19 crane working at cope level.

Water was encountered over the whole area of the extension, the inflow being approximately 350 gallons per minute. This was led to a central sump and delivered into the main dock drainage system by two 6-inch-diameter centrifugal pumps.

Demolition of the original 3-foot-6-inch-thick concrete arch end wall was facilitated by the use of hydraulic cartridges, a process which eliminated the possibility of damage to ships in dock and to completed work.

The mix adopted for all concreting was :

6·75 cubic feet of 2½-inch graded aggregate
3·4 cubic feet of sand
1 cwt of Portland cement,

and was handled into position from two 14/10 mixers located at the head of the dock.

The original concrete floor was cut back square across the dock, and concreting of the new floor, which was 5 feet thick at the centre with a 6-inch cross-fall and a longitudinal fall of 1 in 300, proceeded westwards from the new head wall.

A system of 18-inch-by-18-inch rubble drains was laid under the floor and led to pressure relief valves at the bottom of the walls.

Side walls were of a cross-section shown in *Figs 13* and the head wall was segmental in plan, constructed to a 58-foot internal radius, 4 feet thick at the base and with a vertical face. The portions of the walls

below rock level were drained to the pressure relief valves previously mentioned. These came into operation immediately the final sump was sealed off.

Slight discharges of marl were observed to be taking place from the valves in the early stages after completion, but this later ceased. During this period, one of the valves became jammed in the open position and at high water the discharge from it reached a height of about 2 feet 6 inches above the top of the valve, or roughly 9 feet above underside of floor level. Any increase in this head brought into operation the valves at the base of the head wall, which, owing to later changes in design, were set about 2 feet higher than those in the side walls. The discharge from all valves increases noticeably when the neighbouring No. 1 Dry Dock is flooded. This no doubt results from a closure of the pressure relief valves in that dock and the diversion of ground-water through fissures to No. 2 Dock.

Services

Existing services were taken up and re-laid, rail and road approaches were re-aligned, and the track of the existing 40-ton travelling crane on the north side of the dock was extended on piles and columns for a distance of approximately 35 feet.

Cost

The approximate final cost of the work described was £49,000, of which about £3,500 represented the increase in the cost of labour and materials during the period of construction.

The basic rates of pay at the time of tendering were 2*s.* 4*d.* per hour for labourers and 2*s.* 10*d.* for craftsmen, so that the work executed can be said to have cost approximately 390,000 labourer-hours.

The volumetric unit cost was about 6*s.* 8*d.* per cubic foot measured between walls and from cope to floor level.

Fitting-Out Quay Extension

The existing fitting-out quay lying to the north of No. 2 Dry Dock was originally 550 feet long and was extended, during the course of the reconstruction works described, by 210 feet, to a total length of 760 feet.

As in the case of the lengthening of No. 2 Dry Dock, the work of extending the fitting-out quay continued while the original portion remained in service. A view of the completed quay is given in *Fig. 14* (facing p. 323).

Site Conditions

Boreholes indicated that rock was to be expected at approximately level — 56·00 at the northern end of the existing quay but would be lost

rapidly when proceeding in a northerly direction. Overlying strata were shown to be assorted sands, marls, and sandy gravel, as a result of which it was decided that some form of piled structure would prove to be the most economical type of construction. The heavy swell which was frequently experienced in the area made it desirable for special fendering precautions to be taken in an effort to reduce maintenance costs.

Description of Work

The quay extension, 50 feet wide, consists of six longitudinal rows of vertical reinforced-concrete piles grouped in bents 11 feet apart. The front, middle, and rear pairs of piles are 16 inches, 15 inches, and 14 inches square respectively and between the bents three 15-inch-square piles are driven at a rake of 1 in $3\frac{1}{2}$.

The pile bents are capped with 36-inch-by-20-inch reinforced-concrete beams, between which span longitudinal secondary beams supporting the crane and rail tracks, raker piles, conduits, and kerb, as well as an 8-inch-thick reinforced-concrete deck slab.

In-situ walings, 18 inches by 12 inches, were provided at — 5·00, and horizontal sheeting beams, 20 inches by 9 inches, were fixed against the back row of vertical piles above the level — 7·00. Below this level the filling was rock faced and sloped down under the quay to bed level in the berth, which was at — 36·50.

The quay is provided in each bay with a gravity fender consisting of a horizontally hung concrete block weighing in air approximately $17\frac{3}{4}$ tons. These blocks are suspended by $1\frac{1}{2}$-inch-diameter mild-steel chains from a fabricated steel assembly at each pile-and-waling connexion, and from suspension plates secured to the blocks by $1\frac{1}{8}$-inch-diameter through-bolts in steel tubes. A timber-and-steel fender assembly is carried on the face of each block, which has a kinetic-energy-absorbing capacity of 35 foot-tons, when rising 24 inches in air.

Salt-water, fresh-water, and compressed-air mains of 8 inches, 3 inches, and 6 inches diameter respectively are laid along the extension, and service points are provided at three places. Electrical power, lighting, and welding facilities are also provided and a trench to house the electric crane pick-up gear is incorporated in the superstructure.

An anchorage of 200 tons capacity has been constructed on the reclaimed area behind the quay to facilitate engine trials while vessels are lying alongside.

A plan and section of the quay are given in Figs 15, Plate 2, and some details of the concrete gravity fenders are reproduced in Figs 16, Plate 2.

Construction

The reinforced-concrete piles, of which there were 185, had a maximum length of 77 feet 6 inches and weight of 8 tons, and were cast on a prepared concrete bed behind the existing quay. The mix used was:

7·9 cubic feet of $\frac{3}{4}$-inch whinstone
4·1 cubic feet of sand
2 cwt of Portland cement
9 gallons of water.

These quantities gave an average volume of 9·52 cubic feet of concrete, and the average 28-day crushing strength was 4,000 lb. per square inch.

Piles were lifted and stacked using a three-point slinging arrangement. From the stack, piles were offered by a 10-ton Morgan derrick, working on the reclaimed area, to a 60-foot-high Woodfield frame which travelled out over the work on a temporary wooden staging.

Driving was carried out with a 4-ton single-acting steam-hammer; at first an elm dolly was used, but it was later replaced by plastic dollies which proved to be much more economical.

Penetration was generally of the order of 40 to 45 feet below bed level, where sets of $\frac{1}{4}$ inch to $\frac{1}{2}$ inch per ten blows, with the hammer falling 3 feet, were achieved. The maximum working load for which the 16-inch-square piles were designed was approximately 65 tons.

The casting of the deck and walings followed normal procedure except that the inversion of the deck simplified the shuttering problem at the expense of slightly higher pile loadings. The soft fill was also an advantage with regard to the type of work carried out at the quay and, moreover, the services provided alongside could be augmented below ground without difficulty

The concrete gravity fenders were cast in one operation in situ in a position 2 feet higher than that occupied when normally at rest. This allowed additional time for concreting and simplified the work of securing the hanging chains. Lowering was carried out using tackle slung from the pile capping beams.

The form of face fender finally adopted resulted from experience gained when a vessel of 10,000 tons dead weight was berthed in light condition along a portion of the completed quay. The original design incorporated a timber bulge, about 3 feet deep, localized about the horizontal centre-line of the concrete block. The vessel was drawing an average of only 8 feet of water, and on one occasion at high water in a rather high swell the bilge keel descended on top of the bulge, tearing off some of the timber facing and straining the front lower chain anchorages. This can be attributed to exceptional circumstances rather than a criticism of the fundamental design principles of the fenders, for they are found to be most efficient in practice.

In action, the fenders are quite lively in their normal position and can easily be moved a few inches by hand. Provision has been made for suspending hoisting tackle from the superstructure to facilitate repairs or renewals.

Excavated material from No. 1 Dock reconstruction was used as filling behind the quay extension, and protective works were constructed

along the exposed face of the reclamation between the lifeboat slipway and the northern end of the new quay.

Cost

The approximate final cost of the work described was £77,500, of which about £7,750 represented the increase in cost of labour and materials during the period of construction.

The tendered basic rate for labourers was 2*s.* 4*d.* per hour; it can therefore be said that the work cost approximately 598,000 labourer-hours.

The total cost of the gravity fenders was about £11,000, which represents approximately £52 10*s.* per linear foot of quay extension, whilst the whole cost of construction including rail re-alignments, reclamation, and dredging works was about £7 7*s.* 6*d.* per square foot of deck.

Acknowledgements

The Author is indebted to the Consulting Engineers, Sir William Halcrow & Partners, and to the owners, T. W. Greenwell & Co. Ltd, for permission to submit this Paper.

The Consultants' Resident Engineer was Mr F. S. Maconachie, M.I.C.E., assisted from time to time by Mr D. E. Crook, B.Sc., Mr J. B. Dalman, B.Sc., Mr R. A. Jennings, B.A., A.M.I.C.E., and Mr G. A. Robinson, B.Sc., who ably dealt with all site problems.

The civil engineering contractors were The Demolition & Construction Co. Ltd, for whom the work was planned and organized in London by Mr R. McC. Briggs, B.Sc., and Mr F. J. Watkins, M.I.C.E., whilst on the site Mr. R. H. Rowe was their Agent and Mr M. L. Wolfe-Barry, B.A., A.M.I.C.E., their Chief Engineer.

Vickers-Armstrongs Ltd, designed and built the gate in collaboration with Sir William Arrol & Sons Ltd, who designed and built the hauling equipment.

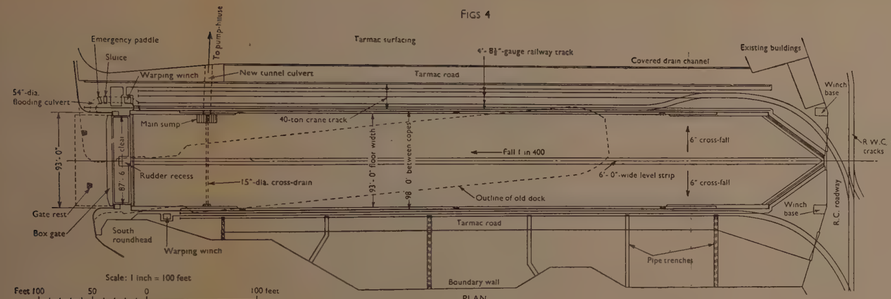
The Author, as representative of Sir William Halcrow & Partners, was responsible for the design and the supervision of construction, and is indebted to their staff and in particular to Mr E. I. Loewy, B.Sc.(Eng.), A.M.I.C.E., whose unfailing energy appreciably simplified those duties.

The Paper is accompanied by six photographs and ten sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

THE RECONSTRUCTION OF GREENWELL'S No. 1 DRY DOCK AND ANCILLARY WORKS AT SUNDERLAND

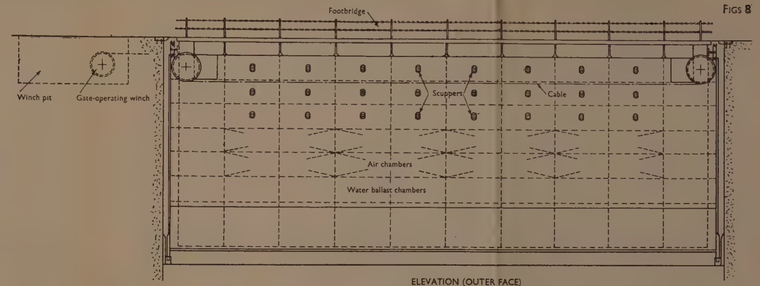
PLATE I
RECONSTRUCTION OF
GREENWELL'S No. 1 DRY DOCK

FIGS 4

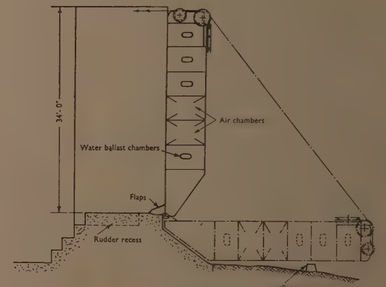


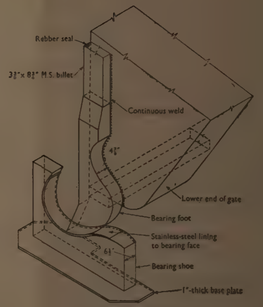
PLAN AND SECTIONS OF NO. 1 DOCK

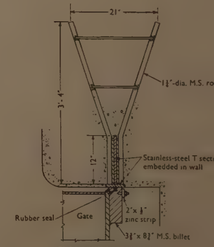
FIGS 8

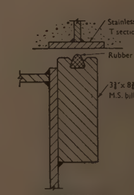


BOX GATE OF NO. 1 DOCK



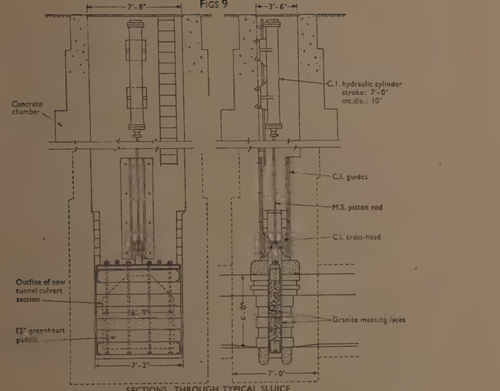




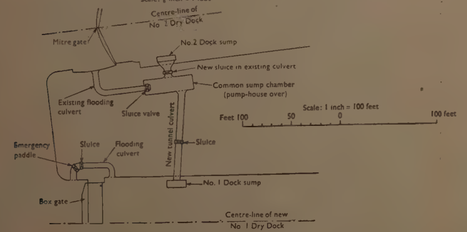


THE RECONSTRUCTION OF GREENWELL'S No. 1 DRY DOCK AND ANCILLARY WORKS AT SUNDERLAND

FIGS 9



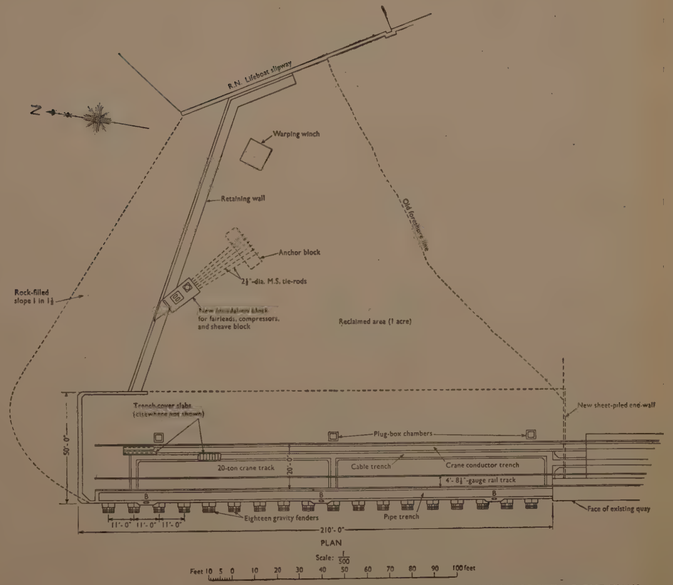
SECTIONS THROUGH TYPICAL SLUICE
Scale: 1 inch = 1 foot



ARRANGEMENT OF CULVERTS, SLUICES, AND SUMPS FOR DOCKS No. 1 AND 2

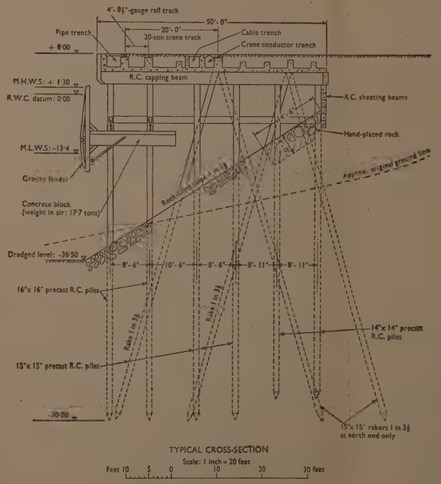
CULVERTS, SLUICES, AND SUMPS

FIGS 15



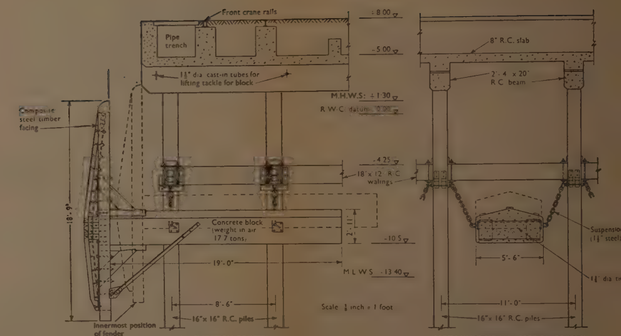
PLAN
Scale: 1/500
Feet 10 5 0 10 20 30 40 50 60 70 80 90 100 feet

EXTENSION OF FITTING-OUT QUAY



TYPICAL CROSS-SECTION
Scale: 1 inch = 20 feet
Feet 10 5 0 10 20 30 feet

FIGS 16



PART SECTION OF JETTY SHOWING ELEVATION OF FENDER UNIT

PART LONGITUDINAL SECTION OF JETTY SHOWING CROSS-SECTION OF FENDER UNIT

ERRATUM. For "Innermost position of fender" read "Partially retracted position"

CONCRETE GRAVITY FENDER

HARRY RIDEHALGH

WILLIAM CLOWES & SONS, LIMITED

Discussion

Mr J. H. Jellett observed that the Author was to be congratulated upon the elimination of the greenheart keel, to which he had referred on p. 331. Mr Jellett had been associated for the past 5 or 6 years with the Dover Train-Ferry dock gates which contained that feature, and there was no doubt that it was one of the weakest features of the design as provided at Dover.

Members of the Division might remember that in 1948 he had presented a Paper describing the maintenance operations at Dover.[1] Several speakers had been inclined to be somewhat critical because the gates had been taken out for attention so recently after they had been put in—for it was within about 10 years. He did not know what those speakers would say when he told them that the gates now had to be taken out again after only 5 years. That had been entirely on account of the excessive wear of the greenheart keel.

Mr Jellett displayed a slide showing a cross-section of the greenheart keel. There could be seen the face which rubbed in contact with the cast-steel horizontal quoin every time the gate was rotated. A small clearance was provided between the steel rib carrying the greenheart and the surface of the steel casting. The timber had now worn down so much that that clearance had completely disappeared and the steel rib was binding on the steel casting. As a result, the gate, as it came up, developed a series of sharp vibrations at intervals as the rib caught on the steel and then slipped past it, a most undesirable result.

First one gate and then the other had been taken out to remedy that, and it had involved rebuilding the entire greenheart keel. The second gate, after having had that treatment, was due to go back in about a fortnight's time.

The debris guard, which was mentioned on p. 333, was also an excellent addition which he wished was installed at Dover. The gates lay flat while any traffic passed over them into the dock, and they acted as an efficient catchment or tray for all the debris that the ships insisted on knocking off themselves and the quayside fenders when they came in. When the gates were lifted into the closed position, all the debris rolled down the face and a large proportion of it found its way into the hinge. When the gates were taken out for repairs, a large assortment of bolts, spikes, and washers were picked out of the face of the timber keel and a monkey wrench had been found which had been cold-rolled to the radius of the quoin.

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[1] "Maintenance Operations 1946–47 on the Box Gates at Dover Train-Ferry Dock." Maritime Paper No. 11, Instn Civ. Engrs, 1949.

An attempt had been made to incorporate the Author's idea, and a guard against debris had now been provided, but since it was impossible to get to the bottom of the dock in the same way as he could, it had had to be very much a second-best arrangement. A small shelf of trough section had been attached to the face of the gate as near to the hinge as possible, and it had been hoped that that would succeed in catching a large quantity of the debris as it came down the gate. There was, in fact, evidence that it did so; about once every 2 months a diver went down to the gate which had been fitted and, on an average, he took a good bucketful of spikes and bolts out of the trough on each occasion.

Incidentally, in his remarks about the Falmouth gate, the Author had referred to the fact that it was docked after 4 years. In view of the critical remarks which had been made on the occasion of Mr Jellett's own Paper, he felt he was justified in asking the Author whether there was any reason why that gate had had to come out quite so soon.

Mr P. A. Scott observed that the history of Greenwell's dry docks, or of the dry docks built on land which was now occupied by Greenwell and Company, went back a long way. As long ago as 1897 a scheme had been prepared for a dock 700 feet long by 72 feet entrance width practically on the site of Greenwell's No. 2 Dock. Greenwell's themselves had built their No. 2 Dock in 1925, and 20 years ago, in 1932, they had asked the late Mr Gedye to report on a project to build a new and larger dock on the site of the old No. 1 Dock. A further report had been made in 1933 on the basis of a dock 515 feet long with a 76-foot entrance width, but it was not until 1939, when Greenwell's and the River Wear Commissioners had obtained a private Act to allow the River Wear Commissioners to lease No. 1 Dock on long lease to Greenwell's, that designs had been prepared for the reconstruction. The outbreak of war, however, had prevented the work from being carried out.

In 1943, No. 1 Dock had been badly damaged by enemy action and rendered unserviceable, and the late Mr Gedye had begun to prepare contract drawings for the reconstruction, but the Admiralty had withheld approval and it was not until 1946 that permission had finally been obtained.

By that time the dimensions of the Dock had increased to 575 feet with an 80-foot entrance width. Tenders received for the construction of that Dock had been found to be unsatisfactory, and before a contract could be let the 1947 Economic Survey and restrictions on capital development had caused the Admiralty once more to withdraw their support.

Greenwell and Company, however, had been undaunted and had continued to negotiate for permission to rebuild the Dock. After 2 years they had been successful, and the work described by the Author had resulted.

The successive delays had certainly been a source of great annoyance, as well as loss of revenue, to the owners, but one beneficial result of those delays had been that today Greenwell's had a dock capable of berthing

32,000-ton tankers, which none of the previously designed docks could have done. Indeed, even that dock had started life with a length of 650 feet and an entrance width of 85 feet. When construction was well under way, the owners had learned of the projected new tankers and had asked for an entrance width capable of accepting 84-foot-6-inch-beam vessels. (Some people would have said that, with 3 inches on either side, that was enough!) The design adopted for the box-gate meeting faces had been found to allow an extra 2 feet 6 inches, and the necessary alteration had been made, giving an entrance width of 87 feet 6 inches. Soon after that, the tanker owners had announced that the new tankers would have a greater length than had previously been allowed for, and, almost at the last minute, the design of the head wall was altered from a semi-circular shape to the present tapered form, giving an additional 25 feet length, to a total of 675 feet, and, incidentally, reaching the utmost limit physically possible on the site.

Whilst the final cost of the dock was much greater than it would have been had it been built in 1932, or 1939, much of the additional cost was more apparent than real and was caused by devaluation of the pound. A cubic yard of 1932 concrete was still a cubic yard of concrete in 1952, but a 1953 pound was worth only a fraction of a 1932 pound. What really counted was that Greenwell's had today got a dry dock at a cost appreciably lower than any rival firm could normally hope to build for. Whilst, as engineers, they naturally claimed some responsibility for that happy result, it had to be admitted that the circumstances of having an existing dock already excavated from the dumpling of the new dock and the fortunate geological conditions had been the principal contributing factors.

While on the subject of finance, Mr Scott noted that on p. 326 the Author had mentioned that had the commencement of the work been delayed for about 6 months, the construction period might have been reduced by the same period, with possible financial advantage to the owner; but against that possible saving should be set the fact that by making an earlier start some of the work had been completed before wages and material costs rose.

On the same subject, on p. 335, the Author had given the volumetric unit cost of the dry dock as about 3*s.* 5*d.* per cubic foot measured between walls and from cope to floor. At the basic labour rate of 2*s.* 4*d.*, that could be expressed as 1½ hour of a labourer's time per cubic foot. As a rule of thumb, that was an interesting figure, and it would be even more interesting if other members could give comparative figures for the cost of other dry docks known to them.

There was, unfortunately, only a very-small-scale site plan of the dock reproduced in the Paper, but by referring to *Fig. 1* and also to the aerial photograph (*Fig. 2*), the extremely congested condition of the site could be appreciated. The centre-line of the dock had had to be chosen with the greatest care, since the entrance position was dictated by the proximity of the oil tanks and 40-ton crane track on the north side, and by the

entrance to the South Docks on the south side. The centre-line of the entrance was therefore dictated within narrow limits, as was the western extremity or entrance of the dock. The angle of the centre-line was dictated by the northern corner of the Corporation Quay seen in *Fig. 1*, and whilst clearance for ships using the dock could have been provided by swinging the eastern extremity of the centre-line further southwards, that could only have been done at the expense of the River Wear Commission's Building at the south-east end of the area. As it was, maximum use was made of the southern area available and some encroachment on the River Wear Commission's property had been necessitated, as had been described in the Paper. As had been proved at the opening ceremony, the alignment chosen was perfectly satisfactory for berthing the largest vessels which the dock could take, and it was to be hoped that the position would be made still more satisfactory at some future date when the present very awkward entrance to the South Docks would be remodelled—it was hoped that that would be done when the expense of the work was considered to be justified.

The original fitting-out quay had been so short that large vessels had had to lie with their stern covering the entrance to No. 2 Dock, and that had naturally been a considerable disadvantage. The reason for the shortness of that quay had been a desire on the part of the late Mr Simpson, who had been responsible for it, not to project into the shipping channel with a solid wall. In addition, Mr Simpson's development plan for the Port at that time showed training walls running diagonally from the inner piers to a point roughly where he had stopped his fitting-out quay, and he could not logically, therefore, have extended his quay any further. That particular plan had now been abandoned, and although the quay extension now encroached to a certain extent into the channel, it left ample navigating space for vessels using the port, and the open-work construction did not form the obstruction which a solid quay would have done.

The Author had referred on p. 338 to the heavy swell which was frequently experienced at the fitting-out quay. The whole question of the entrance to the port had been under consideration for several years, and model-tests were still being carried out to ascertain to what extent and in what manner the entrance piers should be altered to provide calmer conditions within the breakwaters and in the entrance channel, and the design of the new quay extension had been taken into consideration in that work.

Mr Austin Kelly said that he wished to say a few words on the novel form of flat gate described in the Paper.

As had been said by Mr Jellett, the principal advantage which that gate had over the orthodox gate was that the long circular timber sill and the cast-iron socket embedded in the masonry in which the sill revolved were eliminated. Incidentally, the vertical heel-posts on ordinary mitre gates might be eliminated with advantage on new construction.

It would be appreciated that in a gate with steel-to-steel meeting faces the watertight seal was obtained entirely by the rubber insert. That only projected something like $\frac{1}{4}$ inch over the steel face, and it required very accurate workmanship indeed to ensure that those two faces, measuring 90 feet by 35 feet, were constructed with sufficient accuracy to produce the watertight seal. In the present case, they were successful and the gate was a good fit, but if it had not been he was afraid that the work involved in making it so would have been very costly and difficult.

He put it forward for consideration that if, in a gate of that type, the steel-to-steel meeting faces had been replaced by greenheart timber with a rubber insertion on the gate itself, and good-quality concrete with reasonably smooth surfaces on the side and bottom sills, certain advantages would accrue. First, the initial seal would be obtained with the rubber insert as before, but under pressure the greenheart and concrete would form quite a good seal. In other words, they would be providing two watertight seals instead of one, and, in the event of damage to the rubber, the gate would remain reasonably watertight. Secondly, the expense of fitting stainless steel into the masonry would be avoided and the consequent galvanic corrosion which had occurred would be eliminated. Another point was that if the shape of the gate was initially inaccurate, the timber would be easier to make fit than steel billets.

With regard to galvanic corrosion, it was gratifying to learn that in the case of the Falmouth gate it was so slight. Various reports had been received, some of them implying that it was of a much more serious nature.

The fitting of zinc strips was the normal practice in shipbuilding, where " zinc protectors " were fitted adjacent to the propellers and sea-valves and other non-ferrous fittings on the shell of the ship, and they protected the steel shell of the ship in the vicinity. When the ships were in dock, those fittings were examined very carefully and replaced when necessary. In the present case, where the corrosive action was so very slight and the zinc strips were so generous in size, he felt that it would be many years before the zinc would require to be replaced. Incidentally, another way of dealing with it might have been to put cast iron instead of stainless steel into the masonry of the entrance, since cast iron stood up to that sort of work very well.

With regard to the stepping of the gate, he considered that there was little doubt that the surging to which reference had been made was produced by the movement of the flooding water across the gate. However, in his opinion, so long as the top corners of the gate were adequately secured with suitable wires the surging was of no great importance since one always had the gate under control.

In the ordinary course of events, the necessity for stepping would arise at intervals of very many years, and on the initial stepping the whole operation had been completed within 3 hours. Consequently, the expense involved in getting greater sub-division and the fitting of extra flooding

valves, vents, and so on, in order to shorten the time slightly seemed to be hardly justifiable, but, if it were required, that would be the way to do it.

Mr F. S. Maconachie observed that in self-defence he might mention that for about 30 years immediately prior to his assuming duty as Resident Engineer at Sunderland in 1950 he had been engaged wholly on civil engineering projects abroad, sometimes on the resident side and sometimes on the construction staff, while at other times the work had been undertaken with Government personnel and direct labour. He mentioned those facts because they might have resulted in the somewhat abnormal viewpoint on which his comments on the work at home were based.

He had managed to find a little word of praise for the contractors. That occurred towards the end of the Paper under the heading " Fitting-out Quay Extension," where the Author had described the site conditions and given a note of the borings, and at the same time under " Construction " the Author had dealt with the pile-driving in general terms.

Referring to the construction from the contractors' point of view, he described the area in which the piles were driven as strewn with buried boulders and old foreshore structures. In the circumstances, great credit was due to the agent and his staff for the manner in which they had handled and driven the piles, which was borne out by the fact that only two piles had developed cracks and had had to be replaced. Those were rather unfortunate cases in that the damage had resulted from the direction of the piles on sloping rock when the piles were nearly in their final setting.

To achieve that satisfactory result, great care had been necessary to nurse each individual pile and to observe it during the period of driving to ascertain whether obstructions which were met could be penetrated without damage to the pile or whether, much to the disappointment of the contractors, it was necessary to withdraw the pile and remove the obstruction.

As to adverse criticism, he wished to refer to the Author's remarks under the heading of " Cofferdams " where he had mentioned the modification in connexion with the south roundhead at No. 1 Dock. That, Mr Maconachie felt, might have been avoided had the Larssen piles forming the dam been driven with a light hammer. That was based purely on site observation since he had had no previous experience of driving piles in Magnesian Limestone. He had had cases where it had been possible for tests to be carried out in the use of hammers, and in some strata he had found that the heavier hammer paid whilst in others the lighter hammer was the better proposition.

He knew that Mr Watkins, who would be taking part in the Discussion, had favoured the use of a heavier hammer as giving less resultant damage to the toes of the piles, and it would be of genuine value if Mr Watkins had time to make one or two comments on the basis for that.

Mr D. H. Little observed that he was most interested in the design

of the walls and floor, and the gate. Vented dock walls and floors seemed generally to be designed somewhat arbitrarily. At the East London Dock in South Africa, for a maximum keel loading of 75 tons per foot, a 4-foot-thick floor had been built. For a battleship dock at Le Havre a thickness of only 18 inches had been provided, and in a post-war design in Great Britain (but never built), for a keel loading of 150 tons per foot, a thickness of 4 feet had been proposed. The Author had not mentioned the keel loading for which his dock had been designed, but oil tankers would probably not reach 50 tons per foot, and a floor thickness of 2 feet, rather than the 5 feet provided, would have seemed ample—particularly since the old dock founded at a higher level had only had 18 inches. The walls for No. 1 Dock were shown as 14 feet thick at rock level—about 22 feet from cope ; those for No. 2 were 11 feet 3 inches thick and the old existing ones were 7 feet 9 inches. Had the latter shown any sign of weakness ?

The amount of water coming from the vents seemed to be greater than in most vented docks, and the record of that experience was most valuable. Mr Little assumed that the valves were of the usual non-return rubber-ball type generally adopted for that purpose ; although they could not fail to relieve the pressure (that was, to open) when the dock was dewatered, he doubted very much that they would close efficiently when the dock was flooded because he felt sure that they would never be kept clean. If, therefore, there was more water flowing from No. 2 Dock when No. 1 Dock was flooded, Mr Little was confident that that was caused by water leaking out of No. 1 into No. 2. The reference on p. 337 to water reaching a height of 2 feet 6 inches was not quite clear. Was the whole dock flooded to that depth, or did only a single spout from one valve reach that height ? Had special precautions been taken with the floor rubble drains to keep concrete out of them when the floor was poured, and had the wall been back-shuttered to form those drains or had they been taken out of the wall thickness ?

It had become established practice with all forms of caissons—sliding, floating, or Box gates—to incorporate rubber as the water seal, but usually the pressure was taken by greenheart bearing on concrete blocks built into the dock. The elimination of the normal continuous greenheart hinge in that gate was a decided step forward, but that could not be said for the use of steel-to-steel meeting faces. Was the steel really stainless—in other words, austenitic ? A shipwright chargehand who had helped to maintain the caissons in Devonport Dockyard for 40 years had informed Mr Little that he had never found anything wrong with the masonry meeting faces of the numerous dry docks in that dockyard. Since it was such a major effort to repair meeting faces in the dock structure, Mr Little felt that it was vital that they should be as sound as possible, and whilst concrete might not be as durable as granite in terms of centuries, yet with relatively soft greenheart bearing against it well-made modern concrete should last many decades ; it was definitely a far more certain proposition than

" stainless " steel. Timber on the gate might need more maintenance than steel, but at least such maintenance could be readily done, and with the rubber gasket acting as the water seal the amount would be much less than where the timber alone was relied upon as the seal.

Mass concrete proportions for the widening of two Naval docks just before the war had been 7½ : 1 by weight and on one of them, at least, that seemed to be too rich. At the large Singapore dock, 13 : 1 mass concrete had been used (faced with a 4 : 1 mix) and that was reputed to be somewhat lean. Mr Ridehalgh's 8 : 1 was therefore probably just right, without facing. Had it been vibrated mechanically and was it faced with a richer mix ? If not, did it produce a really satisfactory wearing surface to the floor, steps, ramps, cantilever altars, and copings ? Mr Little also had had difficulty in getting cube strengths of 2,500 lb. per square inch with mass concrete, but provided that good density was achieved cube strengths were not very important in structures where working stresses did not exceed 25 tons per square foot (400 lb. per square inch).

The kinetic energy of the gravity fenders on the jetty was given as 35 foot-tons, butsince the weight in air was 17 tons and the lift 15½ inches,* the buoyant kinetic-energy capacity would seem to be more nearly 12 foot-tons. Was the tipped slope under the jetty all rock or earth faced with rock, and had it been placed before the piles were driven or after ? If after, had special precautions been taken to avoid damage by impact or induced bending stresses to the piles ? Mr Little assumed that the jetty had been built by land-based plant, and in that case the cost of £7 7*s.* 6*d.* per square foot was not very cheap—unless the reclamation had been extensive. The cost of fendering was quoted as £11,000, which amounted to about £1 per square foot ; but the tidal bracing had surely cost another £1 per square foot and since the only real purpose that served was to support the gravity fenders, it seemed that the full cost of the fendering system as a whole should be put at about £20,000. He felt that when assessing the merits of flexible-fender designs, it was essential that total effective costs should be known, including the additions that the fenders might necessitate in the jetty structure proper.

Mr F. J. Watkins said that the major decision that the contractor had had to take was the type of dam which should be used. First, an arch dam had been considered, but that would have encroached upon the shipping fairway. The other alternatives were a gravity dam or a single-skin dam, The gravity dam had been adopted because a single-skin dam would have required timber strutting and steel shoring which would have had to be altered as the old work was demolished and again as the new work was constructed.

Another point was that with a single-skin dam there would not be the advantage of the weight of filling behind the water faces, which was a

* This dimension has since been deleted from Figs 16, Plate 2, because it did not indicate the maximum rise of the fender.—SEC. I.C.E.

great help in preventing the water blowing under the steel piles. In fact, that point had been emphasized during the construction of the small dam.

The final reason was that, with a single-skin dam, any blow from a ship coming in or out of the docks would in all probability have been disastrous. On a number of occasions the gravity dam had had slight knocks, but none of them had done more than superficial damage.

When the fixing of the south end had been under consideration, as would be seen from *Figs 6* of the Paper, it had been impossible to encroach upon the shipping channel in any way. From the aerial photograph would be seen the two entrances into the wet dock. Unfortunately, the one which was farther away from the work had been out of action, and all shipping had had to pass right by the south knuckle. The contractors had tried to get permission to drive steel piles along the face of that entrance, but even that small encroachment could not be allowed. The only possible alternative had been to build the south knuckle of the dam on its own, and a small box dam had been constructed.

The dam had been pitched and driven, and the excavation carried down to about 5 feet below the existing work, when there were signs of water coming under the west face of the steel sheet-piling. The excavation had been continued to that depth by using steel runners pitched to the profile of the new foundation. A blanket of clay had been laid outside the small cofferdam, but the space had been so restricted that any large quantity would have caused interference with shipping and have been of little use.

It had then been possible to drive the piles down further after the first blow. That had been possible because the rock at Sunderland was in layers with soft marl in between two layers of hard rock, and once the water had started to seep through, it undermined the top layer, and when the hammer was used on the pile again, it was possible to crush through that layer of rock.

When a heavy hammer was used, the pile would pull up on the rock, but if driving was resumed after there had been a seepage of water through the fissures in the lower layers, a further penetration could be obtained using either a heavy or a light hammer.

After the piles had then been driven deeper, the work had continued, pumping out only at low water, but even with that the flow of water had increased, and it had become necessary to abandon the use of the cofferdam. The revised method had then been adopted of driving the steel sheet-piles round to form the face of the south knuckle. Those had been carried round as far as the back of the new dock wall, where a junction pile had been left so that at a later date, when the side walls of the dock were being put in, it would have been possible to continue the piles, since it had been anticipated that there might be water coming through from the South Dock when excavating for the dock walls. Fortunately, that had not occurred.

The abandonment of the cofferdam had not been all loss because it

would have been impossible to drive the face piles without removing the obstruction inside the small cofferdam.

The question of the size of steam-hammer to use on steel sheet-piling was one that was always open to discussion, but in Mr Watkins' opinion the best results were obtained by using heavier hammers except in freak ground conditions. The heavier hammer should be used up to the point where the heads of the piles began to show signs of damage; once that occurred it was safe to assume that the toes would suffer a similar fate and driving should be continued with the lighter and faster type of hammer, relying on the chipping action of the faster blows.

There was, of course, no guarantee that the lighter hammer would drive the piles any further, but it was always worth trying, as had proved to be the case at Sunderland; he had had a similar experience recently when driving in hard clay on the Medway, whilst in chalk at Seaford the lighter hammer had been useless and had damaged the pile head. Sunderland was a special case because there were layers of soft material between the limestone which tended to wash out (as in the south roundhead); that enabled piles to be driven down after water had seeped in, although previously the piles had been driven to practical refusal. He considered that the unusual result obtained at Sunderland might have been due to the alternate layers of limestone and soft material which, under the heavy blow, would compress and act as a cushion, whilst with the light sharp blows the limestone would have chipped because the force would not have been sufficient to compress the underlying material.

That was a very problematical suggestion and he would like the Author's views on the subject.

Mr M. L. Wolfe-Barry observed that on p. 329 the Author had said that the excavation had been won by R.B.19 shovels or skimmers. That was only partly true, since the bulk of the excavation had been taken out by the R.B.37, to which the Author had referred elsewhere; the R.B.19's had been used mainly at the bottom of the slope of the dumpling.

The cross-section of the double-skin dam had been developed in consultation with the piling firm, and it had been designed to withstand a head of 40 feet. That was much greater than it had ever had to support except in one corner of the sill, because the rock between the dam and the sill had not been excavated when the dam was in place, but had been dredged out afterwards when the dam was being removed. All the same, the rock level had not been quite as shown in *Figs 6*. The level shown was true for the southern half of the dam where the old dock entrance was, but where the dam overlapped the old north roundhead, which had been demolished later, the rock level had been about 6 feet lower. The filling in the dam had been assumed to have an angle of repose of 35 degrees, but when the filling was being placed it had been realized that that was rather an optimistic assumption.

The water level in the dam against the inner skin had been assumed

to be —24·00 R.W.C. Datum. The danger of the water level rising in the dam had been foreseen, and weep-holes had been provided on the inner skin. The weep-holes had been burnt in the piles before they were driven, but since some of the piles had not gone fully down to their correct depth, some of the holes had been rather high when the dam had first been dewatered.

An inspection eye had been provided to keep a watch on the water level as well; the water level in the eye had been observed daily and had nowhere greatly exceeded —20·00 R.W.C. Datum, and it had been thought that the water level on the inner skin had been about 4 feet lower, judging by the level at which it had come through some of the holes in the dam.

Measurements had also been taken of deflexions of the dam at the top waling level, which had tended to show that whereas the northern half of the dam had been deflected inwards, the southern half had shown a tendency to move outwards. Whether that had been caused by the large force of water on the north face he did not know; it was a possibility but it seemed unlikely in a structure of that size.

The dam had originally been intended to extend on the north end to the end of the clay pockets, but if that had been done, the dam and the dolphins protecting it would have hampered docking operations at No. 2 Dock, and it had then been decided to omit the end compartment and to strut that corner off the old quay wall. The puddle pockets had then been an essential part of the design, although the Author had stated that they were only a supplementary part. Nothing had been known at that stage about the gap in the new north wall and it had not been realized that the piles could be driven through it, but even when those piles had been driven, the puddle pockets had played their part. The other small square pocket, shown in *Figs 6*, had also been filled with clay, and he was sure that that had helped to seal the place where some of the piles which were meant to go through the gap in the wall had failed to go down because they had been obstructed by concrete.

The same was somewhat true of the puddle pockets on the south side, where a subsidiary pocket had been made between the piles of the limpet dam and the piles of the main dam. The bottom of that had been filled with clay. That corner of the dam had been very wet, and it was in there that the blow had occurred. If it had not been for the two puddle pockets outside, the blow might have been much worse—perhaps so bad as to be uncontrollable.

Mr E. I. Loewy observed that there were always choices and compromises which had to be made—every job had them—and for the people on the work who could follow the development of those choices, each step seemed logical enough, but to someone who had not been a party to the development of the scheme, the final result was occasionally curious. That made it difficult to apply the designs with merit in another scheme where the circumstances might be rather different.

The first important point he wished to raise was the choice of a vertical face for the walls, with only cantilever altars instead of the more usual battered or stepped walls with altars stepping forward from the top to the bottom. It was true that dry-dock profiles had been getting steeper in the past 50 years, roughly in step with the changes in the shape of ships, but so far as he knew, the wall completely vertical from top to bottom had found favour only on the Continent. In the present case, the designers would not wish to claim too much credit because, to a certain extent, it had been forced upon them. It might almost be said that there had been a sort of conspiracy between the parties to make sure that a quart—the dock, getting bigger all the time—went into a pint pot, or the site, which had not been getting bigger at the same rate! The shipowner wanted bigger ships all the time; the ship repairer, who had to accommodate them, told the consulting engineer that it had to be done—and the consulting engineer would not say that it was impossible. That had led to the use of a vertical wall face, and by curious irony or by the perversity of the laws of nature, it appeared that a wall had been evolved which was more satisfactory for dock operation, gave more room on a restricted site and, at the same time, was very economical in materials.

The resulting altar width of 2 feet might be judged to be rather narrow. He supposed that the chief defence was that the dock owner had had 25 years' experience of another dock with altars even narrower than that, which did not appear to have given anybody any trouble.

He did not want to conflict with the replies by the Author, but Mr Little had implied that the floor was on the thick side. That had rather queered his own pitch because he had thought it might be said that it was on the thin side. There had been the particular experience on the No. 2 Dock which Greenwell's had built in 1925, designed by Mr Simpson. So far as he (Mr Loewy) could see, Mr Simpson had tried to build it as a watertight dock, making no provision for vents of any kind. It was understood that the floor had cracked very soon after the dock had been brought into operation—not disastrously, but it had cracked—and the repair measures taken had been to drill holes through it into the rock and grout in long rail anchors, and to drill weep holes, making it partially anchored and partially vented. In the present case it had been desired to vent the dock before the trouble arose. There was reason to believe that drainage flow would not be very serious because the drainage of the older dock had not been a big problem, and so it had proved.

The designers had wanted to hang the ball valves facing downwards, but the makers had said that that would not work, and so in the end the ball valves were put facing upwards. He did not believe what the makers had said. He would like to see the ball valves hanging downwards; then they would not stick so readily, as Mr Little said they were liable to do.

Mr Loewy supposed that the most important thing to mention in relation to the quay was that, contrary to what Mr Little said, the walings

had not been put in to secure the fenders. The walings were a bit of a nuisance, but they had been considered necessary anyway to stiffen the structure, and had merely had to be put as high as possible to make the swinging fenders fit into place. Therefore, he did not think that they could be brought into the cost of the fendering.

Mr T. N. C. Bulman, referring to the shoe upon which the dock gate revolved, said that he could not make out whether the metal-to-metal faces rubbed one against the other or whether they rolled one over the other rather like the teeth of a gear. If it was the former, he was surprised to hear it. He thought it would have been designed for the latter movement.

In Figs 4, Plate 1, was shown the design of the cross-section of the dock. His first question concerned the uplift; since he was not a dock engineer, he had to try to think those things out for himself. Reference was made on p. 329 to the pressure relief valves; as he understood it, the water pressure in the ground was relieved through those valves and the water going into the bottom of the dock was drained and pumped away. The design of those valves was not shown, but he had had some experience of ball valves, and they did stick. It seemed to him that there was a good chance of water pressure building up in the ground and pressing underneath the dock. Had any preliminary borehole explorations been made over the site by means of which could be ascertained the possible volumes of water which would make their way into those valves? If so, he took it that the number and positions of the valves followed from the information so obtained. Finally, in view of the possibility of the valves not working very well, had any provision been made for reading the water pressures outside the structure at intervals?

At one point in the Paper the Author had stated that a shaft had been sunk and a small tunnel driven two ways, but he had given no further information about that. Mr Bulman would like to know through what strata the tunnel had been driven and how the excavation had been effected; if it was in rock, by what forceful methods the rock was removed; and he would also like to know how the water in the tunnel and the shaft had been dealt with, for no doubt it was very wet there.

On p. 330 the Author had referred to the concrete and said that the relatively small quantities of concrete to be placed in one operation had made central mixing out of the question. Mr Bulman begged to differ on that point. It seemed to him, from the design, that a central mixing plant would have been efficient, and the concrete could easily have been distributed from it to both the walls and the floor, by pump or some other method, and, therefore, the various mixers here and there, each with its own pile of sand, stone, and cement, need not have existed.

Mr G. A. Wilson said that one of the most interesting matters arising in the discussion had emerged from Mr Scott's remarks and concerned the continual increase in the size of ships, which did not seem to

be planned and often occurred half-way through a job. Some of those present might know that the Permanent International Association of Navigation Congresses had recently asked for a study of ship sizes to be made in relation to harbour dimensions and he would like to see some progress made with that, although it was a very difficult subject. The size of tankers already seemed to have surpassed the dimensions of Greenwell's dock, and similar events had taken place at the Eastham Oil Dock of the Manchester Ship Canal. At Manchester, it appeared to be particularly upsetting because he understood that there had been a complete exchange of information with the tanker companies before the work commenced.

The new gate was very interesting and the omission of the greenheart keel was undoubtedly a great advantage. Perhaps he ought to say " promised to be a great advantage," for the new design had yet to be proved in operation. The figures which had been quoted for the intervals elapsing between repairs on box gates were interesting. The Port of London Authority had a large number of mitre gates of the usual type and many of them had been used for 20 years without repairs. That period, which was too long, had been protracted as a result of the war, and very severe wear had been found when it had been possible to undertake repairs. He hoped to submit to the Institution a Paper concerning lock-gate repairs fairly soon. It was usually found that the greenheart heel-post had worn 2–3 inches down to the bolt heads which were rubbing on the granite. He sympathized with speakers who would prefer concrete or granite to stainless steel, and regretted that granite was today so costly.

Another matter which might be of interest to dock designers had been noticed in one of the dry docks of the Port of London Authority, where drainage culverts of 2–3 feet diameter had almost been choked by paint which had dropped on the floor of the dock from hull painting and washed off into the draining channels. He did not know if it was a common experience but it would be well to make provision to avoid it when designing dock drainage.

Mr J. S. S. Ramsay said he had been interested to hear Mr Jellett speak of the excessive wear on the greenheart keels of the Dover gates and of the bucketfuls of spikes, bolts, and wrenches which the divers picked up periodically, and which obviously were the cause of the excessive wear.

On p. 332 the Author had stated : " It might here be remarked that, as a consequence of the new design, the work of setting the meeting faces and bearing shoes was transferred from tradesmen to engineers, with undoubted advantage." His experience did not lead him to corroborate that. There were some engineers who were all right on roads and sewers, but when it came to meeting faces on dock gates he would prefer a good tradesman ! He had certainly seen excellent results from them.

Referring to the tackle arrangements for stepping the gate as shown in *Fig. 12*, he observed that all the lines in the diagram made it appear

very complicated and elaborate, until one sorted it out and found that the majority of them were arrows pointing to the various tackles! The stepping of a gate was really a very simple process, and sometimes all that elaborate tackle could be modified if the stepping were done at high tide, which was the most suitable time.

He did not think that a gate could be kept under control when being up-ended with a rope attached to each of its four corners. In the case of a gate 90 feet long, with no centre bulkhead in the watertight compartment which was used for up-ending the gate, he would not feel that he had control of it if he tackled it with only a couple of ropes at the top. A centre bulkhead in the water-ballast compartment was essential.

Reference had been made to using the rubber seal with greenheart against a concrete face. The rubber seal in metal had really been introduced in order to avoid the very fine and expensive work of dressing the faces on the granite and the greenheart. It was difficult to get men with experience of that kind of work. In the intermediate groove in the Sturrock Dock at Capetown there was a concrete face, and a rubber insertion had been put in the greenheart on the caisson. In the outer grooves there were granite faces. There was no difficulty in setting the stainless-steel strip on the concrete face; it just wanted special care. There was no great difficulty in getting the meeting faces watertight when a rubber insertion was used.

Mr V. H. Pontin said that on p. 330 the Author had stated that it had been the original intention to use hydraulically operated sluice valves but it had been found impossible to obtain such equipment in the time available. Could the Author state what type of gear was actually used?

*** **Mr R. McC. Briggs** observed that Mr Maconachie was of the opinion that if a lighter hammer had been used on the south knuckle piles, penetration could have been reached such that the south roundhead, as originally designed, could have been constructed. Mr Briggs was doubtful of that, since fissured crystalline formations were most frequent in that corner. While taking the piles in the main dam to the depths eventually reached, driving had been virtually at a standstill for about 2 feet or more, and 350 blows per inch had been constantly recorded. That had been after the south roundhead experience. When driving (using a heavy hammer) had been stopped on the roundhead, for practical purposes a standstill had been reached. The toes of the piles had then been at about formation level (—40·00 O.D.) and considerable doubt as to the efficacy of the dam had been expressed at that time. By stepping in to the profile of the footings once rock was reached when excavating it had been thought that the piles at that depth might be effective. A cover below formation of about 5 feet was thought to be the safe limit, which had meant that a further 5 feet of driving was necessary if security could be reasonably assured. That was not apparently possible though, perhaps, with steady

*** This contribution was submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

driving with a No. 7 hammer such cover might have been reached at great expense in time. When the blows had occurred, the weak spots had been marked and the piles had in fact been driven down at those spots about 2 feet. That had been after the blows had occurred, and in his opinion the extra depth of driving obtained had been achieved only because the " fines " had been released and it had been possible for the driving to break the hard strata between the soft layers once the soft layers had been weakened. Unfortunately such further driving had not been effective after the protection had been weakened by the loss of the fines. To have driven the piles to proper depths in the first instance would have been a formidable task and not reasonably to be considered. As it was, the piles had been damaged and one or two had been out of interlock. With greater space around the dam more might have been done, but in the very exacting conditions involved, to face up to the likely heavy pumping below the level of the footings of the adjoining exposed entrance wall would have been a grave matter. That had been appreciated by the Consultants when the change of design had been agreed to. Such change was only practicable, as Mr Watkins had observed, because where the masonry and concrete of the old wall had been on the alignment of the new roundhead they had been successfully removed behind the roundhead dam before it had failed. That had allowed permanent piles to be pitched in the rock below the old wall to the profile of the new.

Reference had been made to the concreting arrangements and the possibility of central mixing. Mr Briggs wished to say that comprehensive plant arrangements had been considered, with central mixing, derricks, and loco tracks, but it had been very obvious that such a lay-out of plant would not be economic owing to the difficult conditions with only scattered short lengths of site available. The method adopted, using highly mobile plant and local mixing of concrete, had been found to be the most effective. A more stereotyped arrangement of plant at the entrance would have been preferred, with derricks commanding all the work in that area, had space permitted. Greenwell's 40-ton crane was a fixture in the entrance area (until it had been made mobile by the construction of the new crane track later in the job) and had defeated any more satisfactory plant arrangement, although very considerable use of it had been made. There had been no direct access to the south knuckle until very late in the contract, and all the initial work there had had to be handled from the north wall.

The Author, in reply, said that it had been considered necessary to take the Falmouth gate out after a period of 5 years in order to keep a close watch on how the new design was behaving. No defect had, in fact, been observed, other than very minor electrolytic action on the faces of the meeting billet. It should be appreciated that Mr Jellett's congratulations on the design of the gate had really been earned by Messrs Arrol's and Vickers Armstrongs, who had prepared the designs and carried out the construction.

Mr Scott had referred again to unit prices. The Author wished to emphasize that the figure of 3*s.* 5*d.* should not be taken as an absolute criterion for dock building. As he had said, the last estimate he had prepared for a new dock had worked out at 11*s.* per cubic foot which, for another rule of thumb, was something like four times the present hourly rate for labourers.

It had been foolish of him to say that there might have been a saving in cost if the Contractor had started 6 months later. What was meant was that if the Contractor had had a clear run at the site he could probably have organized himself better. Whether he would have charged the owner less or not, the Author really did not know ; he very much doubted it !

Mr Austin Kelly had re-introduced the " hardy annual " of meeting faces. The Author did not think that whatever he said would alter the opinions of other people. Though at one time he had supported the use of concrete for meeting faces, subsequent experience had altered that opinion. He believed that a tight face could be obtained by a rubber insert in greenheart, but that would not get over the difficulty encountered if the greenheart sustained damage. In his opinion the experience gained with the Falmouth gate over a period of 5 years was enough to prove that it was a very good design ; in fact he thought it was the best design that they had had so far. Mr Wilson appeared to doubt that, but the Author considered that if a new gate lasted for 5 years and nothing had to be done to it when it was taken out for examination, then there was nothing much wrong with it.

He was sorry that Mr Maconachie thought that he had not given the contractors credit in connexion with the piling on the fitting-out quay, but he had been told that in the Paper he had said far too much in favour of the contractors and so thought that he could be excused in that instance.

He was surprised that Mr Little had criticized the thickness of the floor. He did not think that a thinner floor would have been justified, in view of the particular type of rock encountered. Its bearing capacity varied quite rapidly but the Author agreed that, had it been a good igneous rock, then a thin skin of concrete would probably have sufficed.

Even though keel loading of tankers might not exceed 50 tons per foot, one had to remember that it was not uncommon for ship-repairers to remove several adjacent keel blocks at a time to facilitate plating. The Author did not consider that the walls as designed were excessively thick.

A certain amount of trouble had been experienced owing to the ball in the pressure relief valves becoming stuck, but that difficulty appeared to have been overcome by increasing the diameter of the outlet holes. The reference to water reaching a height of 2 feet 6 inches was to a single spout from one valve which had become jammed when the dock was dry. The rubble drains behind the walls had been cut out of the rock, and all floor rubble drains had been carefully protected during concreting to

prevent debris from being washed into the rubble filling. Concrete generally had been vibrated and the walls had not been faced with a richer mix.

The Author apologized for a discrepancy in Figs 16, Plate 2, in the Advance Copy of the Paper * which had mislead Mr Little regarding the kinetic-energy capacity of the fenders. The blocks in fact lifted 2 feet when fully retracted, giving a capacity of 35 foot-tons in air. The bracing on the piles for the fitting-out quay had been put in to reduce the unsupported length of piles as columns, since they were rather long. The bracing was conveniently placed for hanging the fenders and the Author thought that the length of chains used was just about right ; if the chains had been any longer the fenders would probably have been too lively.

He agreed with Mr Watkins that the use of a double-skin cofferdam had been correct. A single-skin dam would have proved difficult to keep water-tight, but it would no doubt have been cheaper and quicker had it been practicable.

The contractors had been obviously happy when the design of the south roundhead had been changed. That had got them out of a hole, both literally and metaphorically, but he still believed that it would have been possible to build the original subsidiary cofferdam, and in view of the experience since gained he would not do anything different from what had been proposed originally, if he had to do it again.

The remarks which had been made by Mr Wolfe-Barry were useful in amplification of what was given in the Paper about cofferdams. The Author was sorry that he had not been able to differentiate between the R.B. 19 and the R.B. 37, but since those machines were often transformed overnight perhaps Mr Wolfe-Barry would forgive him for that error.

With reference to the remarks by Mr Loewy, he considered that vertical barrel walls were the answer in that particular case. However, he would not necessarily always put in vertical walls ; their use would depend on whether vessels using the dock would be of fairly constant beam or not ; if of uniform dimensions, then the walls would probably be better stepped in order that ventilation and lighting of the dock bottom was not restricted. If a vertical wall was required in such a case, then it would have to be sited a good deal farther back from the line of the entrance walls, with the disadvantage that the distance between copes would be unnecessarily increased.

In reply to Mr Bulman, the Author confirmed that the metal-to-metal faces of the gate hinges rubbed against each other and the unit pressure did not exceed 0·5 ton per square inch. The only indication obtained of any pressure built-up behind the walls was, as stated in the Paper, that water had spurted out of one of the relief valves which had jammed. That showed that the pressure on the underside of the floor was equivalent to a head of about 10 feet of water. The tunnel had been driven through

* See footnote, p. 348.

soft rock by hand tools and was timbered, whilst the water had been carried away in a drainage channel to a central sump.

He still did not think that central mixing would have been a good thing. The contractor had been given possession of the whole site at irregular intervals and it would have been uneconomical to work a centrally disposed mixing plant.

Mr Wilson's remarks about ship sizes were pertinent, but the dock could take the biggest tanker afloat at the moment. They had not yet built a dock to take a tanker of 44,000 tons, but he believed that he was correct in saying that such a vessel was not yet afloat.

Despite Mr Ramsay's remarks, the Author considered that he would prefer to have an engineer with a theodolite setting the meeting faces, rather than a tradesman. As stated in the Paper, he thought that a centre bulkhead in the watertight compartment was desirable, and agreed that greater control of the gate during the stepping operation would be afforded by the use of additional control wires.

The Author stated in reply to Mr Pontin that the operating mechanism of the penstocks was hydraulic, but the paddles themselves were made of greenheart.

He considered that the question of the most suitable hammer weight for pile-driving, which had been raised by several speakers, was one best decided by actual experience on the site. He was quite satisfied that the day was passed when a contractor could send to the site the first hammer available from his stock and hope to drive the piles successfully. Whilst a light hammer had undoubtedly proved successful at Sunderland, he had two cases in mind where the equally successful driving of piles had been concluded by using a heavier hammer than that initially intended.

Correspondence

Mr D. G. McGarey referred to the statement on p. 333 that all steelwork of the dock gate had been pickled in a 5-per-cent solution of hydrochloric acid, and that, after erection, protection had been given to the various surfaces by the application of bituminous and grease paints. He presumed that the pickling had been carried out on the component plates and sections before assembly but after cutting to size, drilling, and shaping.

In view of the size of the main components and the fact that a 5-per-cent solution of hydrochloric acid had been used, it could be assumed that the treatment had been by the cold bath process, involving periods of immersion of up to 16 hours, as compared with 10–15 minutes where a stronger 10- or 15-per-cent solution could be used in a heated bath. Would the Author say what facilities had been available and the approximate size of the largest pieces treated ?

There was no doubt that pickling gave an excellent basis for protective treatment of steelwork, but it was very desirable that the first protective

coating should be applied as soon as practicable after the pickling process. A phosphoric-acid wash assisted in retarding the onset of the inevitable subsequent rusting of the pickled surface, but normally a primer of red lead or other suitable composition would be applied within a matter of hours. In the case in question it would appear that a long period must have elapsed before post-erection treatment with bituminous solutions, etc., and it would be helpful to know what precautions had been taken to ensure that the beneficial effects of pickling were not lost by reason of surface rusting during that period.

In any case, one would have expected that advantage would be taken of the ideal opportunity offered by the pickled steel surfaces to give the maximum possible protection by applying a good priming coat of genuine red lead, or perhaps one of the alternative primers that were nowadays recommended in some quarters. The subsequent application of bituminous solutions and enamels or grease paints would then have provided the desirable second line of defence and at the same time acted as a protection to the priming coat. The length of time necessarily elapsing between the application of the primer before fabrication and the bituminous coatings after erection would have been sufficient to ensure that no trouble was likely to arise from " bleeding."

Finally, it would be of interest to have some further details of the grease paint used for the interior surfaces of the water compartments and the reasons for adopting that type of paint. Some—if not all—types of grease paint did not harden-off completely on drying, and one would perhaps expect that the coating would be liable to eventual damage and breakdown in those particular conditions. Was it not thought that the impervious qualities of the bituminous coatings would be suited to the water compartments ?

Mr R. A. Stephenson observed that a severely restricted site and enforced short-term planning were typical of dock reconstruction in general, and, in those cases where the building owner was a port authority of some considerable size, such circumstances constituted, according to a well-known argument, one of the main factors favouring the direct-labour system ; the reasoning was, of course, that the authority had only to stand the bare cost of the unavoidable inefficiences, whereas with the contract system the authority might have to pay for contingencies that never occurred. Having in mind the Author's experience of both systems, Mr Stephenson asked how he would view that argument in the light of his latest experience ?

The Author's description of the rock encountered showed it to be far from ideal material for single-skin cofferdam working. No doubt it shattered considerably as piles were driven into it, and perhaps that accounted for withdrawal not being as difficult as might have been expected from the penetrations quoted ; that was, if the rock outcropped at original bed level as Mr Stephenson assumed.

He believed that sealing the joints between blocks of mass concrete by means of some form of water-stop was probably superior to leaving short gaps between the blocks to be concreted later but without sealing the joints. He had heard of cases where noticeable cracking had still occurred with the latter system. Had any cracks developed in the blocks between joints at Greenwell's ? The form of water seal adopted was rather neat. Had it proved fully successful and what was the approximate cost per foot run ?

Mr Stephenson would like to know the general design of the pressure relief valves and the approximate head necessary to open them. Presumably the object of the valve was to prevent the ingress of silt when the dock was full, so that the vents would not become choked.

The type of bearings adopted for the Box gate was remarkable for the reduction of bearing area achieved. What was the maximum preponderance of the gate and the corresponding bearing pressure ? Presumably the bearing foot was so shaped that no part would project above sill level when the gate was open. Or was there some other reason why it was not extended through a further 90 degrees to cover the contact surface of the shoe at all times ? One of the main hazards associated with Box gates was the danger of silt or debris preventing the gate from opening fully. Had any special precautions been taken against that contingency ?

The method of estimating the proportion of hydrostatic load taken by the decks and by the bulkheads was always of interest in such structures, supported as they were on three sides. Of the conventional methods known to Mr Stephenson, that which assumed the decks were elastic beams propped by rigid bulkheads hinged at sill level would appear to be the best applicable to Box gates. Was the Author in a position to say what method was actually used ?

The dimensions of the lower end of the hydraulic-sluice-gear shafts enabled an economical form of cross-head to be fitted, but was not the space available for access in the case of emergency rather restricted ? Was the mild-steel piston-rod provided with a corrosion-resistant sheath or was that precaution considered unnecessary ?

The extension to the fitting-out quay provided a number of interesting points. The inverted beam-and-slab construction appeared to be a compromise between the normal beam-and-slab arrangement and a thick slab. Mr Stephenson would expect that, compared with a thick slab, there would be some saving in concrete but little, if any, in steel or dead weight. To what degree had the soft fill been compacted and how had it been surfaced ? Also, what mooring-post pulls had been allowed for in the design ?

How had the shuttering supports been secured to the concrete piles ? Had a friction grip been relied upon as the sole means of support ? Would it not have been preferable to have used precast walings with only the end connexions cast in situ, or was the River Wear comparatively clear of silt ? In any case, would it not have been cheaper, in view of their being located only a foot or so above half-tide level ? Further, could not means

have been found of providing diagonal bracings raked up to the capping beam from the second and rear piles of each bent, thereby enabling a considerable number of walings to be dispensed with, especially since the unsupported lengths of the third, fourth, and fifth piles of each bent were hardly enough to warrant the provision of lacings below high-water level in order to reduce the cross-section required ?

Mr Stephenson was not absolutely clear about the final form of the facing of the gravity fenders. He presumed that the original timber bulge at the horizontal centre-line had been retained unaltered and that it spanned practically the full width of 5 feet 6 inches of the concrete block, thereby serving to spread the impact over at least two ship's frames ; also that the ends were curved back to deflect horizontal blows from vertical projections on ships' sides, much in the same manner that the batters of the three vertical rubbing pieces deflected vertical blows from horizontal projections. That arrangement would appear to provide maximum protection for both ship and fender. He would suggest, however, that the bulge of the fender face was placed a little low, since he would prefer the impact to take place above the water-line whenever possible. The fender units seemed to be spaced remarkably closely in view of their cost. Was that done with the object of reducing vulnerability to small craft such as tugs, or was it the result of some bitter experience that the Author had had ?

Mr R. C. Harvey confined his remarks to the entrance and gate. He was particularly interested in the use of rubber on steel for the seal at the meeting face and thought that that type of watertight joint would be used more extensively in the future. It appeared that the rubber seal was held in position in its trapezoidal recess by a degree of initial compression with or without the use of an adhesive. Would the Author give some further information on that matter and state the mechanical properties or specification of the rubber ?

Regarding the surge of the gate as it was being trimmed from the horizontal to the vertical position, Mr Harvey said that the conclusion that shorter water-ballast chambers would have reduced that was probably correct. He had, however, found that it was necessary to provide many escape holes for both air and water around the perimeter of intermediate bulkheads or diaphragms if tanks were to be filled or emptied steadily. With the old type of riveted construction such holes usually occurred as a matter of structural convenience, but with welded construction one had to make sure that enough were put in.

The Author had considered the matter of the corrosion of the gate very fully, but Mr Harvey considered that the replaceable zinc strips could with advantage have been fixed a little further away from the particular face of the mild-steel billet which they were expected to protect, as was usual in ship-building practice. In fact, he suggested that instead of using the zinc strip for that limited protection, magnesium anodes could have been provided—probably at no extra cost—for the cathodic protection of all

the underwater steel at the entrance. Such protection would, of course, be additional to that given by the bituminous and paint coverings, but it would ensure that rusting did not take place at any small spots which might have been left bare. Anodes for the protection of the gate itself might be located in the harbour bed near the entrance sill and beneath the gate when the latter was in the horizontal position. Electrical connexion to the gate would be by V.I.R. cable of sufficient length to allow the anodes to remain on the sea bed when the gate was in the vertical position. An arrangement of that kind would give an added degree of anti-corrosion protection to the inside surfaces of those water chambers which were flooded through the grated openings.

The remarks made on p. 332 of the Paper regarding the work of setting the meeting faces and bearing shoes having been done by engineers instead of skilled tradesmen, were in accordance with the present-day tendency to arrange engineering work so that mainly " unskilled " workmen were used. Mr Harvey thought that although that trend might be inevitable because of economic and other reasons, it was regrettable and not in the best interests of civil engineering, which would always be an art as well as a science and so would always need craftsmen. He hoped that the engineers engaged on those important and precise parts of the works described in the Author's very interesting Paper on present-day commercial dry-dock practice had in fact made use of some tradesmen, if only to fix the piano wires !

The Author, in reply, confirmed that pickling had been carried out on the component plates and sections before assembly and that immersion of plates up to 38 feet 7 inches by 109 inches had continued for 15 hours. Unless some protective coating was applied soon after pickling, rusting would ensue, but the main purpose of the pickling was to remove mill scale and the subsequent removal of light rust was a comparatively simple matter. The Author did not entirely agree that the immediate application of a priming coat was a good idea having regard to work (including welding) which had to be carried out. The comparatively small rust deposits subsequent to pickling were removed by wire brushing prior to the application of bituminous and grease paints.

The grease paint used was " Camrex " No. 5, applied hot, and whilst it did not go off hard as in the case of an enamel, the dock owner's experience had led him to prefer grease paint in the water compartments.

In reply to Mr Stephenson, the Author stated that he had a personal preference for direct-labour work where it could be backed by a sound works organization and staffed with the right people. It was impossible, however, for non-civil engineering companies economically to maintain and operate such organizations and the contract system was the inevitable answer. Furthermore, since tenders were received on a competitive basis, the possibility of paying for contingencies which never occurred was reduced to the absolute minimum.

There was no cracking in the wall blocks themselves, and the water seal, which cost approximately 25*s*. per linear foot, was very effective.

The pressure relief valves were a proprietary article manufactured by Messrs Glenfield, Kennedy & Co. Ltd, and the Author thought they could be improved by the use of a more open form of cover. Inverted valves would also prevent jamming of the ball.

The outer end of the gate when in the down position was supported on stools about 2 feet high and the sluicing action caused by lowering the gate was, in the Author's opinion, sufficient to prevent the accumulation of silt.

The gate had been designed assuming the decks and a proportion of the skin to act as beams spanning between meeting faces.

The Author agreed that the space in the sluice shaft had been reduced to the absolute minimum and confirmed that the connecting rods had been "fescolized," by which process a thin nickel film was deposited.

Pulls of up to 50 tons had been allowed for when designing the fitting-out quay, which, where not concreted, had been surfaced with rolled ashes.

The Author thought that whilst the use of precast work had many substantial advantages the design of the substructure of the fitting-out quay was, considering all circumstances, the most satisfactory.

Mr Stephenson's interpretation of the fender design was correct and the spacing of the actual units had been dictated by the abnormal conditions of swell experienced at the quay, as well as to guard against the possibility of the structure itself sustaining major damage.

The Author, in reply to Mr Harvey, confirmed that the rubber insert on the gate was held in position in part by a small degree of initial compression and then by the undercut shape of the housing. The rubber was manufactured by The Dunlop Rubber Co. Ltd.

The suggested use of cathodic protection for the gate was interesting but the Author doubted whether dock owners could be persuaded to maintain such a facility. The renewal of zincs was something with which they were all familar as part of their normal ship-repairing activities, and it was felt that the use of zincs was preferable in that instance.

Correspondence on the foregoing Paper is now closed and no further contributions will be accepted.—Sec. I.C.E.

RAILWAY ENGINEERING DIVISION MEETING

27 January, 1953

Mr J. C. L. Train, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Railway Paper No. 48

"The Design of Structures in Relation to Maintenance and Inspection"

by

Frank Turton

and Norman Stewart Cox, B.Sc.(Eng.), M.I.C.E.

Part I.—Bridges

by

Mr Turton

SYNOPSIS

Part I of the Paper sets out to show that the relationship between the maintenance and inspection of bridges and their design is a very close one, and that design should always be directed towards simplicity and the true functional use of materials with a view to facilitating inspection and reducing future maintenance to the minimum.

The seriousness and magnitude of the maintenance problem is discussed and the importance of routine inspection as a part of the maintenance organization is stressed.

Factors giving rise to maintenance work are considered, and reference is made to several points with regard to cost of maintenance.

Various aspects of design based on the preceding premises are then considered. It is argued that most railway-owned bridges should be regarded as "permanent" and that the real cost of a new design should include its maintenance in perpetuity as well as initial cost. Erection costs are also considered.

Changing conditions are discussed in relation to precise theoretical design and the saving of small quantities of material.

The suitability of various types of bridges is indicated, and attention is drawn to the importance of the choice of materials. Reasons are given for the use of various materials in differing circumstances.

Introduction

The cost of maintenance work on bridges owned by British Railways is high and is continually increasing. Not only is the unit cost of labour and materials rising, but the volume of work required to be done is increasing. Deterioration becomes more pronounced with the passage of

time, as factors which hitherto have been dormant gradually make themselves felt. It seems unlikely that there will be any real alternative to railways for many years, and indeed from the engineer's point of view it is reasonable to assume that they are "permanent." Since bridges are an essential part of the railway, it follows that they also must be regarded as permanent and must be maintained in a fit state to carry traffic safely.

Many bridges have been reconstructed, and no doubt many more will have to be, but the rate at which reconstruction can be carried out (taking into consideration such items as availability of staff, labour and materials, and finance) is comparatively slow, and even if it were doubled it would result in only a small minority of bridges being so treated in a reasonable period of time. Also, if it were possible to attain a much higher rate of reconstruction, a serious problem would arise with regard to speed restrictions and line occupations. It seems certain that the majority of existing bridges will have to be preserved, and that the maintenance problem will eventually reach huge proportions and will make corresponding demands on revenue.

In addition to the direct cost of bridge maintenance, the cost of disorganization of traffic resulting from speed restrictions and line occupations necessary for the work to be carried out must be borne in mind. Also, in addition to cost, inconvenience to the travelling public caused by the late running of trains when speed restrictions have to be imposed is a factor which should not be overlooked.

A careful consideration of these points leads to the conclusion that new bridges and the reconstruction of existing bridges must be designed to minimize the future expenditure of labour and materials on maintaining them, in order to permit the maximum use of available resources in preserving older structures.

The Author has purposely limited the scope of this part of the Paper to small- and medium-span bridges, for they constitute the major problem of the bridge engineer on British Railways. Large-span bridges are rarely constructed in Great Britain, and, indeed, in such cases it is very likely that, the design being regarded as a special problem, maintenance will also be treated in the same way.

Inspection

The inspection of bridges is really a part of their maintenance—indeed, it is true to say that an efficient maintenance organization depends upon efficient inspection. Quite apart from considerations of safety, inspection should act as the intelligence upon which future programmes of maintenance work are based. Inspection is the key without which the machinery of a maintenance organization cannot be set in motion, unless it is merely provided to function when something goes wrong. It is, however, neces-

sary to recognize that inspection is of two kinds : one is concerned with seeing that the structure remains in the condition in which it was handed over to the maintenance organization ; and the other is concerned with what shall be done with defects and when. The first is the routine inspection by appointed examiners, usually artisans, who may find defects which they will report upon, and thus bring into action the second function of inspection. This is the inspection carried out by supervisory or technical staff, from whose reports and recommendations programmes of work are prepared.

The importance of the routine inspection cannot be over-emphasized, for if defects are not observed at this stage, it is possible that the first intimation of anything being wrong may be the failure of a member, if nothing worse. This point should be given serious thought by designers and detailers, and especially should it be noted that the routine inspection of the structure which will eventually materialize from their working drawings will be carried out by men who have perhaps had a very limited technical training.

A further aspect of routine inspection which deserves consideration is the very human one that, since there are so many bridges to be inspected, there is a tendency to give scant attention to a new bridge, which may be regarded as " new " for many years.

The cost of inspection can be heavy, especially where it is necessary to erect scaffolding, or to have parts removed in order to enable proper inspection to be carried out. These points should be carefully considered by the designer, and particular attention paid to providing reasonable means of access to all parts of the structure which cannot be reached by portable ladders ; also the design should be such as to avoid the necessity of having to remove anything in order to see what is happening to a hidden part.

Maintenance

The extent to which maintenance work is required depends upon the influence of a variety of factors, either individually or in combination with each other in a destructive capacity. Individual factors are not always easily diagnosed, nor are they all fully understood, but their effects can be seen and it is essential for the designer to study those factors most commonly met with if he is to avoid these effects. In addition to those in the list which follows there may be others peculiar to the site, and it should not be overlooked that some factors will act in combination and that certain combinations will be far more harmful than others. This point may be of great importance when the designer finds that he cannot achieve his ideal on some particular design, and consequently has to effect a compromise. Some factors may be beyond the direct control of the designer, but he should, nevertheless, be aware of them and should endeavour to

take them into account, within reasonable limits, when deciding his safety margins.

Factors affecting the life of a bridge

Live load. Wind. Water. Engine smoke and blast.
Atmospheric pollution. Temperature changes.
Faulty design and detail. Inferior materials.
Bad workmanship. Faulty (neglected) maintenance. Subsidence caused by mining.
Accident. Carelessness. Wilful damage.

The effects of most of these factors can be appreciated from a study of past experience, but the part played by live load alone is somewhat problematical.

Although the applied live load, including all impact effects, may be well within the carrying capacity of the bridge, calculated in the usual manner, it is not known if the life of the bridge would be shortened by the continual intermittent application of such a load. In the present state of knowledge of the subject it can only be assumed that a bridge of near-perfect design, well maintained, would continue to function satisfactorily until the whole, or part of it, had received sufficient applications of live load to cause fatigue failure. If this is true, and the designer pays due regard to the information which research on fatigue has so far placed at his disposal, it would seem reasonable to suppose that a bridge designed to modern standards of loading would suffer no ill-effects on this account alone, provided that axle loads remain sensibly as they are today. But if other factors are not given equal attention in the design stage and during construction it is possible that the continual application of live load will contribute seriously to the shortening of the life of a bridge.

If a bridge is badly designed or detailed it is probable that several of the other factors will combine to hasten its end. A designer might consider that he had produced a good design, and it might be equally well detailed, but if it were of the wrong type to suit the prevailing conditions it would be faulty and would suffer according to the degree of departure from the ideal. It is possible that a type selected without due regard being paid to all factors which might give rise to maintenance work on the bridge itself will also be troublesome from other points of view. The permanent way engineer or the road engineer might be put to unnecessary trouble and expense in maintaining the permanent way or the road surface as the case may be.

Much of the expense of maintenance work derives from difficulties encountered in gaining access to the affected parts, not only because of the possible necessity of erecting costly scaffolding, but because of the need to avoid whenever possible any interference with the running of traffic. The painting of steelwork, for example, in close proximity to running lines, or

over a railway or busy road, might prove extremely expensive, and indeed might be interrupted to such an extent as to render the value of the work doubtful.

Apart from accessibility and interruption from traffic, the cost of maintenance work is also directly affected by the number of parts in a bridge. It is evident that if a bridge were to consist of a single unit of a given material (assuming, for this purpose, brickwork, masonry, reinforced concrete, etc., to be homogeneous materials), it would cost less to maintain than the same bridge in a similar material but consisting of a number of units.

A proper appreciation of all the factors involved in inspection and maintenance will lead the designer to see that they all exert their influence in the same direction, namely, towards simplicity in design and detail and the true functional use of materials.

Design

Cost

The Author contends that maintenance considerations should be of supreme importance to the railway bridge designer and should dominate his approach to every problem. By this it is not meant that every bridge should be built to last indefinitely with little or no attention, nor is it intended to imply that other considerations should be ignored.

The choice of type of a new bridge or a reconstruction will be influenced to a greater or lesser extent by limiting factors which will preclude the attainment of the ideal. It will be a compromise after due regard has been paid to such items as location, function, availability of materials, plant and labour, interference with traffic, and maintenance. A bridge required for temporary purposes, for example, to last 10 years or less, would be of the simplest and lightest type, consistent with safety requirements, and could be designed to be built cheaply and to cost nothing to maintain. A bridge of a " semi-permanent " nature, built to serve for a specific job, the end of which can be foreseen—say, after 50 years or less—would require to be rather more robust in order to avoid possible renewal within that time, and would consequently cost somewhat more than the temporary bridge and would perhaps require some attention during its life. In these examples the cost of ultimate removal should be borne in mind. There are, however, very few railway-owned bridges in Great Britain which could be regarded as " semi-permanent," and it is contended that all, except purely temporary bridges, should be considered as " permanent " and the most suitable type selected accordingly.

If bridges are to be regarded as " permanent," it follows, in considering costs, that first cost is only a part of the problem and that it is misleading to compare designs on this basis alone. The cost of maintenance is as much a part of the real cost as it is in the case of a machine, but, unlike the latter, it should be calculated on a basis of perpetuity.

Comparisons made on these lines lead to the conclusion that cheese-paring to save a little material here and there is not worth while, and that it will not be truly economical to attempt to reduce erection costs if, by so doing, there is a risk of the completed bridge not being entirely satisfactory from the maintenance point of view. For example, a far better job will result if a bridge can be erected and completed before traffic is allowed to pass over it : site connexions, if any, can be given proper attention and waterproofing will be done more efficiently. In some over-bridges the building of a temporary bridge to carry traffic while the new bridge is being built might be possible. For the same reason an under-bridge will be more satisfactory if it can be rolled into position, unless it is a small bridge which can be lifted-in and completed in one operation.

Erection costs of underbridges are usually taken to mean the total cost of all engineering operations, including those of the Civil Engineer's Department, the Signal and Telecommunication Engineer's Department, and the Mechanical and Electrical Engineer's Department, plus locomotive and crane charges imposed by the Motive Power Department; but other Departments also incur costs which should be considered. The Commercial Department costs are, perhaps, intangible since they relate to the goodwill of the public, but those of the Operating and Motive Power Departments, incurred in the slackening of trains for example, can be assessed and should be taken into account when an erection scheme is being planned. If erection costs are estimated in this way it will be found that, where a choice is possible, a scheme entailing a prolonged slackening of trains is often uneconomical when compared with some other scheme in which slackening is reduced to the minimum.

Generally, skew bridges are more troublesome to maintain than square spans and are also less amenable to precise analysis and straightforward detail. Wherever possible they should be avoided, even if the initial cost is thereby increased.

Some designers, whilst acknowledging the maintenance engineer's claims for consideration, nevertheless appear to be preoccupied with producing designs that precisely satisfy their calculations, which have been made to conform as closely as possible to maximum permissible stresses. Whilst not decrying the effort to produce an economical design, it is urged that the designer's ability and skill should be tempered with an appreciation of the possibility of changing conditions. His estimated dead-load figures may be considerably exceeded in the future if more ballast or road metalling is added for some unforeseen reason. Similarly, his estimated total live load might prove to be inadequate 50 years hence. Also, possible deterioration should be taken into account. If considerable margins had not been provided, intentionally or otherwise, in many existing bridges it is probable that the volume and weight of traffic known today would have been impossible.

Since changing conditions are largely matters for conjecture, the margins

which should be provided to take care of them can only be guessed and must be left to the designer's judgement. This is perhaps counselling the practice of " adding a bit for luck," but in fact the cost of a little additional material on the total cost is not great. For example, if a steel plate-girder 62 feet long with a flange width of 20 inches, weighing 21 tons and costing perhaps £1,800, were to have its bottom flange increased in thickness by $\frac{1}{8}$ inch the additional cost would amount to no more than £22.

In considering the cost of a steel girder bridge there are several points that should receive careful study. It is urged elsewhere in this Paper that the steelwork should be welded, but it should not be overlooked that if it is left exposed it might be necessary at some future date to carry out repairs—perhaps to the bottom flange of a girder which may have suffered from corrosion. This would be a more difficult operation than in a riveted girder and, therefore, more costly. In such a case an additional thickness of steel in the bottom flange might well prove more economical. The cost of future maintenance of any type of bridge is impossible to estimate with accuracy, but this fact is not of great importance if the figures are used for comparison purposes, provided similar assumptions are made in each case. The steel-and-concrete bridge, however, requires somewhat greater consideration in this respect. If the steel itself is designed to carry the total live and dead load then the only function of the concrete is to preserve the steel. In some cross-girder floors it is, of course, permissible to reduce the amount of steel by virtue of the distributing power of the concrete, and indeed dead weight in the floor is no disadvantage, but concrete encasement of the main girders without the provision of special means for making the concrete share in the work of the girder can only be regarded as protection to the steel. This means of protection is of doubtful efficacy as well as being expensive in first cost. It is not possible to say with certainty that corrosion does not take place under the concrete and, if it should, remedial measures might prove extremely difficult. In the Author's opinion it is desirable, except in special cases, to leave the steelwork exposed in places where it can be efficiently painted.

In reinforced-concrete bridges particular attention should be paid to the cover of the reinforcing steel. The cost of providing adequate cover is small compared with the cost of remedying defects caused by the cover being cut to the bare minimum.

Type

It has already been stated that the maintenance of bridges does not necessarily end with the structures themselves, and that the permanent way or road surface must also receive consideration. In the case of a bridge to carry the railway there should not be any change in the character of the permanent way, that is, the rails, sleepers, and ballast should be laid over the bridge in the same way as they are laid on the formation on each side of the bridge. Similarly, in the case of an overbridge, the road surface

should pass over it without interruption. Both under- and overbridges should therefore have closed floors and these must be impervious to water, not only to protect the structure, but to prevent damage caused by dripping water to the road surface or permanent way underneath, where they exist.

From the point of view of the permanent way engineer or the road engineer, the bridge superstructure carrying either rail or road should be as unobtrusive as possible. This is a requirement which is also desired by the user, because it is of great importance to him to be able to run traffic without obstruction.

In ideal situations arches more nearly satisfy these conditions than any other form of construction. They have also proved superior to other forms in the past from the point of view of inspection and maintenance. It is the Author's belief that where conditions are favourable to this form of construction the arch should be the designer's first choice.

The arch vault, or sheet arch, has much to commend it. It approaches nearest to the complete unit, and if the spandrels are made solid and the space between them filled with suitable material and the whole carefully waterproofed, with adequate drainage, the type has distinct advantages over all others.

The open-spandrel type of sheet arch is useful where it is desired to reduce the total dead load, or possibly for aesthetic reasons, but it is necessary to provide support for the deck in the form of columns or walls, which may run either longitudinally or transversely. This type is sometimes useful in reconstructing or strengthening existing arch ribs, as shown in Figs 1, Plate 1.

Apart from special types such as cantilever and suspension bridges, which are more suitable for large spans rarely met with in Great Britain, the beam type must be considered. This type of bridge, in its various forms, consists essentially of two parts, the superstructure and the substructure, and the two, so far as stress analysis is concerned, are usually designed as separate entities, although in some cases the effect of friction between the superstructure bearings and the abutments might be allowed in designing the abutments to resist the forces tending to overturn them. In general this is an undesirable practice since it might lead to difficulties should it be necessary to carry out repairs necessitating the temporary transference of vertical load from the tops of the abutments to some other support.

Substructures consisting of mass abutments and wing walls, with the addition of piers in bridges of more than one span, are usually satisfactory if carefully designed and where conditions are favourable. Abutments of this type may be made cellular where the height is so great as to necessitate an uneconomical thickness and the load on the ground may be excessive. Where this course is adopted easy access to the chambers for inspection purposes should be provided, and the chambers should be ventilated and

care taken to prevent the access of water. The drainage of water from the backs of abutments is of the utmost importance. An example is shown in Figs 2, Plate 1.

In certain circumstances substructures composed of trestle bents, as shown in Figs 3, Plate 2, offer considerable advantages over other types, even if in some cases this involves the adoption of additional spans, for it may be possible to eliminate abutments and wing walls.

Of all the beam-type superstructures the best from the maintenance point of view, and the most economical, is the deck type. In small spans it might take the form of slabs, whilst in longer spans longitudinal girders supporting a floor on their top flanges might be adopted. In overbridges of this kind it is usually desirable to connect all the girders together in some way so as to ensure the live load being distributed among as many of them as possible, but in underbridges it is better to arrange for a pair of girders under each track to be rigidly braced together and only connected to the adjacent pair by light supports for inspection walkways, thus permitting free deflexion of the structure under each line. The principle is illustrated in Fig. 4, Plate 2. In both under- and overbridges of this type it is desirable to provide permanent inspection walkways in each bay, and where this is impracticable, provision should be made in the design for some kind of support on which planking can be placed. Access from the top to the main-girder-bearing level should be provided where the bearings are more than 25 feet from the ground, and a clear way behind main-girder ends should be arranged to permit an examiner to enter each bay.

Restricted construction depth often precludes the adoption of the deck type and recourse is then usually had to some form of the through type. The ideal arrangement of through type consists of two main girders with a floor spanning between them, and in underbridges such an arrangement would be provided for each track with, of course, sufficient space between each adjacent superstructure to allow adequate inspection and possible future maintenance. In small spans it is possible to use this type without a space between adjacent superstructures by filling the girder webs as shown in Fig. 5, Plate 2. Provided the span is not too great the superstructure can be prefabricated and erected as complete units for each track. In longer spans the ideal of separate superstructures for each track is rarely achieved, and although in some cases a double-line bridge might have only two main girders with cross girders spanning between them—an arrangement which may be regarded as second best—the more general case consists of three main girders, two outer and one centre, a type which does not lend itself easily to precise analysis.

Cases do arise, particularly in reconstructions of underbridges, where the available construction depth is so small as to leave no room for ballast, and here several expedients have been tried with varying degrees of success. One method is to fasten the rails to a continuous plate which in turn is bolted to the bridge floor, as shown in Fig. 6, Plate 2. There are, of course,

many objections to this from the maintenance point of view, but it is perhaps no worse in this respect than rails supported on longitudinal timbers resting in troughs. A bridge superstructure composed of trough girders is the least desirable of all types, but nevertheless there is often no alternative left to the designer, although some improvement over older types can be effected as indicated in Fig. 7, Plate 2.

Overbridges are generally at no great height above the railway and consequently are exposed to attack by engine blast and sulphurous fumes; this, of course, also renders inspection difficult. Protection of a more or less efficient character can usually be provided, but in order to reduce the effects of attack to the minimum and to ease the problem of inspection, the soffits of overbridges should be flat, except perhaps where they are more than, say, 25 feet above rail.

Materials

When discussing cost it was stated that maintenance considerations should dominate the railway bridge designer's approach to every problem. If he thinks in this way, he cannot fail to ascribe great importance to the choice of materials—indeed, it is possible that his final choice for a particular case might influence his selection of type to a great extent.

The use of stone will probably be confined to localities where it is easily obtained and where the specialized labour is available, and in such cases its weathering qualities can be judged from a study of the buildings and other structures in the district. Bricks, however, are not always so easily judged; they are sometimes obtained from brickworks situated at long distances from the site where they are to be used, and it is not always possible to see how similar bricks have behaved under the atmospheric conditions of the district. Blue and blue-brindled bricks can usually be counted upon to satisfy the most rigorous requirements, but other types may be desired, perhaps, for aesthetic reasons, and in such cases they should be selected with great care.

As structural materials, stone and brick have very limited applications outside the field of arches and substructures of other types of bridges, and some alternative must therefore be considered. In the Author's experience, the material that approaches nearest to good quality masonry and brickwork in lasting qualities, taking cost into consideration, is a high-grade concrete, and furthermore it can be made to overcome the limitations inherent in brickwork and masonry construction. If used in association with steel, as in reinforced concrete, it can, in certain circumstances, be made to serve the same purpose as fabricated steelwork, and if it is prestressed by some means so that it is always in compression under load, it becomes a truly functional material, seriously rivalling fabricated steelwork in a wider field than reinforced concrete, and promises to possess lasting qualities superior to either steel or reinforced concrete.

It must be recognized, however, that one of the constituent materials

of prestressed concrete has developed well ahead of the other. It might be said that steel is the one material in which the structural engineer would place his entire confidence so far as reliability of its properties is concerned. His justification for this attitude lies in the knowledge that if he orders steel to any specification it will conform to that specification within very narrow limits. The reasons for such reliability are many and are the result of long experience, but two of them will serve the purpose of the present argument—research and control. Research, of course, dictates what materials shall be used and in what proportions, and control ensures that what research demands is supplied within practical limits. If steel is to be used to the full value of its capabilities in conjunction with concrete, to make a method of construction which shall possess properties in which the engineer can place the same degree of confidence that he does in steel alone, it follows that the concrete must be manufactured with the same care and under similar conditions. This means that the concrete must be precast in a factory suitably equipped for manufacture within narrow limits to any reasonable specification.

In-situ concrete will, however, continue to be used for a variety of purposes, but no matter what the required grade, the specification should be drawn up with care and the concrete carefully made.

By following the maintenance line of thought the conclusion is reached that prestressed concrete should receive serious consideration by designers in the early stages of every bridge scheme.

Circumstances do arise, however, which preclude the use of concrete in any form, and it may be that steel is the only material which will satisfy the conditions of a particular case. Here, also, future maintenance must be in the forefront of the designer's mind. Bold and simple lines without fussy detail are the first essentials if painting is to be rendered easy and, therefore, of maximum efficiency. Good welded design achieves this object admirably and at the same time shows a substantial saving in the weight of steel when compared with riveted steelwork.

The protection of steel by means of paint or metal coatings, after suitable surface preparation, is a problem which deserves the utmost consideration, but it should not be overlooked that concrete in certain situations also requires protection, particularly where it is exposed to engine smoke or in floors where there is a possibility of injurious chemicals or oil being deposited.

Acknowledgements

The Author wishes to express his thanks to Mr J. Taylor Thompson, M.C., M.I.C.E., Civil Engineer, London Midland Region, British Railways, for permission to present this Part of the Paper, and he also desires to acknowledge the help he has received from members of his staff.

Part II.—Structures other than Bridges

by

Mr Cox

SYNOPSIS

Part II of the Paper deals with railway structures other than bridges and viaducts. The designer's approach to a problem is analysed and compared with the maintenance engineer's demand for ready inspection, cheap maintenance, and infrequent and easy renewals; special reference is made to saving on first cost at the expense of costly maintenance. The value of soil mechanics investigation in connexion with foundation design is stressed, with special reference to settlement.

The influence on design arising from the extended use of concrete is discussed, with mention of certain defects in design of this material; special precautions to be taken in designs using other materials are mentioned.

Facilities for inspection and factors affecting maintenance are enumerated in connexion with drainage, floors, walls, gutters, roofs, smoke chutes, and engine pits.

Introduction

Part II of the Paper is restricted to railway structures other than bridges and viaducts and, although the views expressed will refer to conventional railway works, the terms of reference are yet so wide that it is not possible to cover the subject exhaustively within the limits of a single Paper.

Moreover, whilst there has been progressive improvement in the design of railway bridges to meet changing conditions, there has been less spectacular development in the design of other railway structures, and such changes as have taken place (with the possible exception of those affecting locomotive sheds) are not peculiar to railway work, but conform to developments in general building construction.

The Author, in consequence, has felt obliged, apart from a few brief generalizations on the subject, to restrict his remarks to a limited number of examples of the influence of design on future maintenance, as seen from the viewpoint of the District Engineer.

The first reaction of a District Engineer responsible for the maintenance of structures of many different types would commonly be a general condemnation of the designer for his lack of foresight, the term "designer" being taken to refer to the technical staff of the New Works and Steelwork Engineers or the Architect charged with the design of structures.

Upon reflection, such wholesale criticism would be manifestly unjust, since many engineers and architects responsible for design bring to bear experience of maintenance work, whilst others, lacking that knowledge, have the records of their predecessors to assist them in avoiding the more obvious pitfalls. Nevertheless, many details must be left to the designer

and it is the neglect of these details which may have serious repercussions on future maintenance.

One safeguard introduced by the former Great Western Railway, and which is still the practice of the Western Region, is for drawings of proposed new works to be sent to the District Engineer for comment before they are finally approved. He is thus afforded the opportunity to criticize any features incorporated in the plans which he considers may affect his maintenance adversely.

The designer inevitably approaches the problem from a different angle from that of the maintenance engineer and, in selecting the fabric of the structure and in deciding its form, he will be influenced by the following considerations :—

(1) The function of the structure.
(2) The material most suitable for the fabric.
(3) The availability of material.
(4) Site conditions.
(5) Expectation of life.
(6) Cost limitations.

All too often, the ideal design has to be modified in view of cost limitations and, under present conditions, material shortages. Undue saving in first cost at the expense of future maintenance is false economy and, in the Author's opinion, can never be justified. Again, cost limitations, material, and sometimes labour shortages incline the District Engineer to a preference for the type of structure that is sufficiently durable not to require too-frequent inspection and which can be maintained cheaply and examined with reasonable ease. As the age of the structure increases and inspection becomes necessary at more frequent intervals, these factors become more and more significant.

Another important consideration affecting the cost of maintenance is the facility, or otherwise, with which parts of a structure more susceptible to deterioration than the main fabric can be renewed without undue disturbance of the main structure. Care in the selection of the most suitable material for such parts will determine the frequency of such renewals.

Attention to the design of fastenings cannot be too strongly stressed, since they can affect the life of the structure as a whole, and non-corrosive fastenings of ample section at increased first cost can achieve long-term economy.

The worst types of structure, from the District Engineer's point of view, are those in which shoddy materials are used to save first cost, or those designed for a limited life which he is expected to maintain for an indefinite period. Mass-produced sectional buildings fabricated from unsuitable or even inferior materials may cost as much to erect as properly designed permanent buildings of a better class, and soon become a charge

on maintenance. The resulting differences in cost per cubic foot are generally small and experience shows that, the greater the difference in initial cost, the higher is the subsequent maintenance cost.

Temporary works erected to encourage the development of traffic may achieve their object but, if retained indefinitely instead of being replaced by permanent structures, can in due course become uneconomic.

Site Conditions

On too many occasions in the past the District Engineer has had to contend with settlement troubles, which have arisen from the use of rule-of-thumb methods in the design of foundations. Today, the study of soil mechanics has made such advances that, after site conditions have been investigated by the soil mechanics specialist, much more accurate data is available than hitherto, so that a decision may be made regarding the most suitable type of foundation. At this stage, it is essential for there to be the closest possible co-operation between the designer and the soil mechanics investigator regarding the form, size, and depth of foundation. Since this is usually the one part of a structure which the District Engineer cannot inspect periodically, the importance of sound design cannot be too strongly emphasized, for undue settlement or failure of foundations is very costly to remedy afterwards. The Soil Mechanics Section should also advise the designer what settlement may be expected when the full load is applied, so that he may allow for this in the superstructure, for otherwise it will fall to the lot of the maintenance engineer to make adjustments to maintain specified clearances. The possibility of an artificial lowering of the water table by pumping during construction should not be overlooked.

Equally important from the point of view of economy in future maintenance is the selection of material for the fabric capable of withstanding adverse atmospheric conditions, either natural or otherwise.

Influence of Materials on Design

Concrete

The widespread adoption of concrete, plain or reinforced, to replace masonry, brickwork, timber, steel, and iron might at first sight encourage the belief that this material, which should not require periodical attention comparable to the application of protective coatings to timber, steel, and iron, or pointing, is the perfect answer to many maintenance problems.

Admittedly, the use of this material has resulted in an appreciable reduction in maintenance in some cases but it is not infallible. There is little indication that concrete will be supplanted for many years to come, but greater care in design and manufacture is required, for there are many examples of earlier reinforced-concrete structures—especially those in which competitive design has resulted in skimping—which today show serious defects. In the main, porous concrete and inadequate cover to

reinforcement, coupled with bad workmanship, have resulted in corrosion of the steel reinforcement and spalled concrete.

Heavy maintenance costs will be incurred during the next decade or so in the repair or replacement of many reinforced-concrete structures.

An example of this can be seen in *Fig. 8*, which shows a warehouse of approximately 250,000 cubic feet capacity, where major repairs have had to be put in hand at a cost amounting to nearly £9,000.

Masonry

From the point of view of the maintenance engineer, it is to be regretted that present-day costs and the shortage of skilled masons and stone-cutters preclude the general use of masonry in modern railway structures.

Given sound construction in the first place, the enduring qualities of masonry structures are demonstrated by the number still in use which were constructed a century ago and which call for inspection at less frequent intervals than their steel or timber counterparts, maintenance being limited to occasional pointing and cleaning-down where appearance is important. Furthermore, the well-designed masonry viaduct, bridge abutment, retaining wall, or building has an aesthetic value that few brick or concrete structures can equal. Their main disadvantage arises when changing conditions call for structural alterations, but a similar criticism could be levelled against reinforced-concrete work.

With an eye to first cost, the modern designer can seldom permit himself the luxury of masonry when selecting the fabric of his structure but, on the rare occasion that this material can be employed, he should select his stone with the greatest care, for heavy maintenance can arise from the use of stone which is too soft or which splits or spalls.

Sometimes, the local stone is used to save first cost, regardless of its suitability, and the Author recently experienced trouble with a heavy limestone over-sailing course, from which a piece of spalled stone, 3 feet long, fell and caused considerable damage. It would have been better to have used granite for this over-sailing. (*Fig. 9.*)

Brickwork

To a lesser degree than masonry, brickwork is being supplanted by concrete cast in situ or precast, and the added cost of brick facings to mass concrete in retaining walls can, in the main, be balanced by the saving in shuttering, especially where there are quoins to be formed or re-entrant angles.

Brick curtain walls in reinforced-concrete-framed buildings give the maintenance engineer less trouble than thin concrete panels.

Where appearance is important and the cost of masonry facings prohibitive, a better effect can be achieved with brickwork than with concrete, unless great care is taken and extra expense is incurred in the provision of special shuttering or in surface dressing.

With occasional pointing, brickwork generally maintains a good appearance, whereas concrete tends to suffer discoloration and, in certain situations, appears to encourage the growth of disfiguring green algae. Chemical treatment to remove this growth may be quite an expensive matter.

Timber

Less timber than ever before is being used on railway structures, for the substitutes developed during war-time timber shortages have established themselves, and in many instances their higher cost has been offset by the increase in the price of timber. The world shortage of well-seasoned timber compels the designer to restrict his selection of this material to the minimum.

On the other hand, if the class of timber formerly used in railway structures should again become available at an economic price, there is much to commend it in that, within the limitation of its strength, it can be fabricated cheaply and is economical in maintenance, being less affected by adverse atmospheric conditions than iron or steel. When defects develop in individual members, piecemeal replacement can generally be undertaken without undue disturbance of the main fabric.

An example of this was to be seen in the timber viaduct trestles constructed by Brunel in Cornwall, units of which could be renewed under traffic.

The principal disadvantage of timber is, of course, the fire risk and, for this reason, apart from economic considerations, no substantial increase in its use is to be expected.

Where timber is used, the designer should avoid the type of construction likely to encourage the development of dry rot, that is, enclosed spaces without ventilation ; and the sills of timber-framed buildings should always be laid on brick or concrete plinths well above ground.

An example of the type of construction which resulted in dry rot spreading through a platform building is shown in *Figs 10*.

Steelwork

Mild steel, after ousting wrought iron on the score of increased strength and cheapness, is still the foremost of all materials in use in railway structures, but restricted supplies have in recent years encouraged the encroachment of reinforced concrete.

The standard sections evolved in the course of years of experience lend themselves to design, and this material is economical in first cost even at present-day prices. On the other hand, its maintenance is frequently one of the heaviest items in the District Engineer's budget.

The period of inspection is determined by the rate of corrosion, which can vary considerably in different parts of the country, reaching a peak in industrial and maritime areas.

Excessive corrosion can have such disastrous consequences that its

Fig. 8

Reinforced-Concrete Warehouse in need of Major Repairs

Fig. 9

Spalled Limestone Course

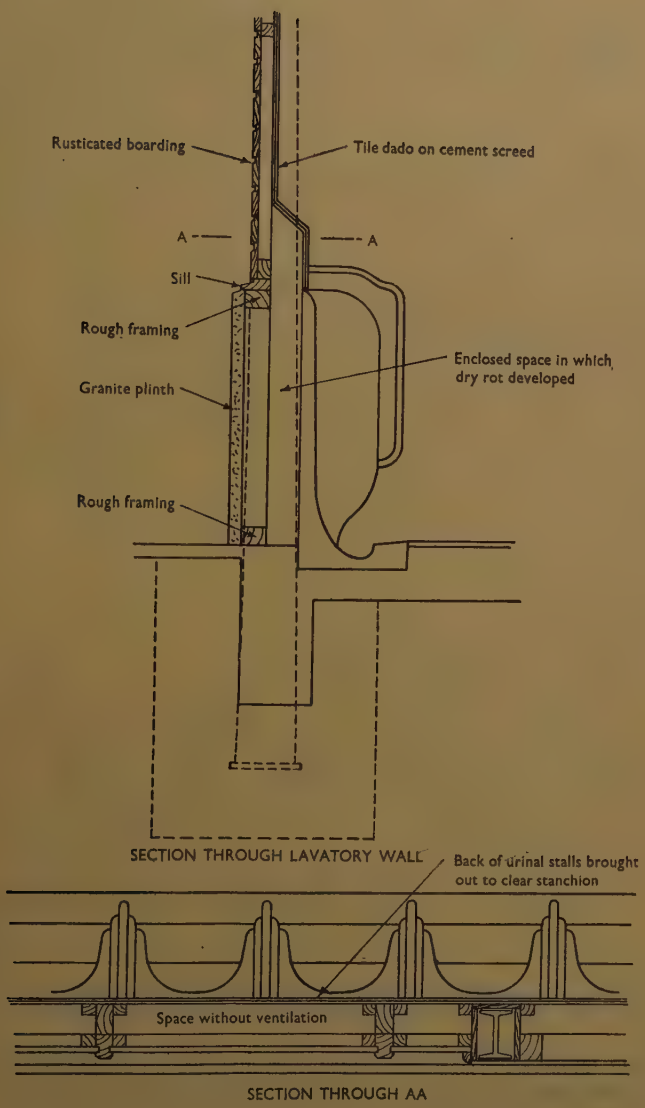
Fig. 19

Smoke Chute constructed with Asbestos Cloth secured in Frames

Fig. 21

The Girderwork of a Bridge kept clear of a Hillside prone to Landslides. (See p. 396)

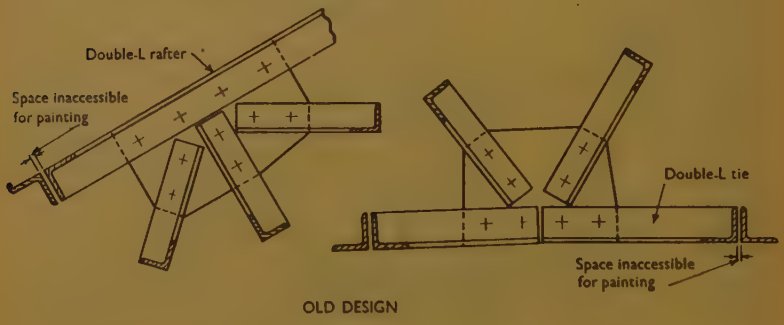
Figs 10



TYPE OF CONSTRUCTION LIABLE TO DRY ROT

early detection is imperative and the designer should, therefore, ensure that all parts of a steel structure are easily accessible. Casing in timber or other material should be avoided so far as possible but, if unavoidable, casings should be readily removable so that all parts of the steel framing can be inspected and painted. It is very important to avoid pockets or narrow spaces between members which cannot be reached by a paint brush.

Figs 11



Roof Truss Details

Figs 11 show a conventional roof-truss detail with angles back to back so close that painting between these units is impossible. An alternative arrangement is also shown, allowing all faces to be painted without difficulty.

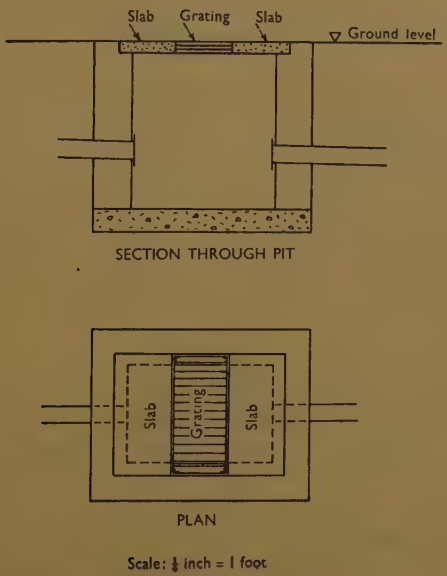
Facilities for Inspection and Factors Affecting Maintenance

Drainage

Inspection manholes should be sufficiently large for a man to operate drain rods when bending down, especially when the manholes are deep. In track drainage, manholes should not be more than 66 feet apart and should be closer in old-type drains laid with open joints, in which the rapid accumulation of silt calls for frequent rodding.

Cover slabs which are too heavy for one man to move unaided restrict inspection, whilst lighter slabs, which can be lifted by the ganger when walking his length, frequently break in handling or when permanent way materials are unloaded at the site.

Figs 12



Inspection Chamber

Steel gratings are preferable in that they are more durable and the condition of the manholes is readily seen so that attention can be given before chokage takes place and the clearance of the drain becomes a major operation.

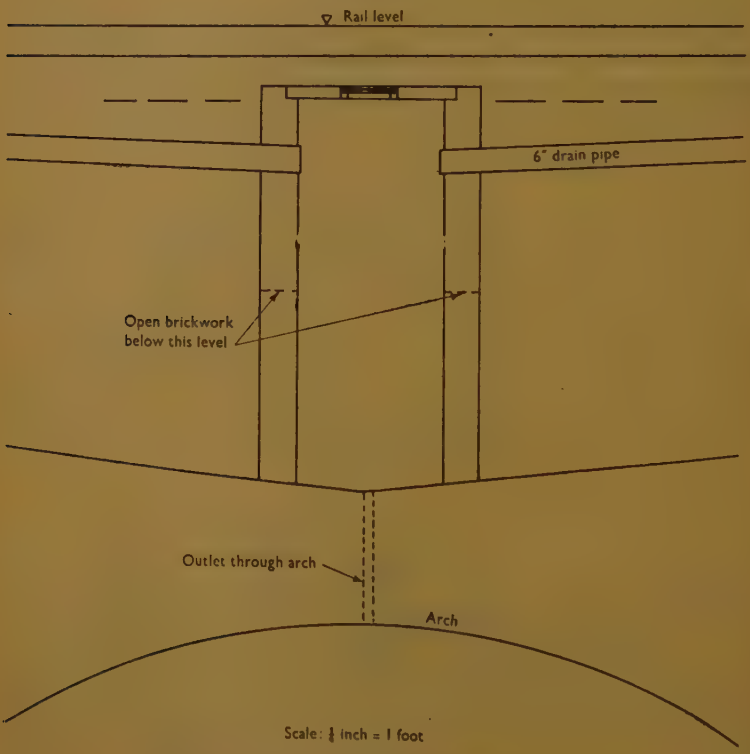
Figs 12 show an inspection chamber with a combined grating and slab cover, but the Author's preference is for gratings extending the full length of the pit.

At the line side, provided there is adequate width for the formation of a cess and the formation is permeable and stable, open channels are preferable to pipe drains and cheaper to maintain.

It is false economy to use pipes of too small a diameter, especially in main drains to which additional connexions may be made at a later date.

Owing to the tendency of outlet pipes in the arches of viaducts to choke, the Author advocates the provision of inspection chambers sited over the

Fig. 13



PROVISION OF INSPECTION CHAMBER OVER OUTLETS

outlets, with open joints in the lower courses. Where there is a considerable depth of filling over the extrados, generally of rubble stone, the chokage of interstices causes the rate at which water reaches the outlets to decrease in the course of time, and inspection chambers as described, with a shallow track-drain system, obviate the necessity of renewing the filling. (See *Fig. 13.*)

Floors

The almost universal adoption of concrete for the floors of railway buildings, with various surface finishes, according to the whim of the designer, does not necessarily solve the problem of floor maintenance. In

goods sheds and warehouses, it is customary to specify granolithic surfacing from 1 inch to 1½ inch thick but, no matter how much care is taken during construction, it is practically impossible to achieve a surface of uniform hardness. Soon pot-holes appear where trolleying takes place and sections have to be cut out and made good. In the Author's opinion, the extra cost of special surface finishes according to the nature of traffic is justified by the subsequent saving in maintenance.

Rock asphalt supplied and laid by specialist firms as a rule stands up to wear better than granolithic concrete, and when the surface eventually breaks down it is more easily renewed.

On shed platforms where goods are unloaded, hardwood boards screwed down to bearers laid in concrete are necessary to prevent surface abrasion from the impact of heavy loads, often with sharp corners.

The designer in future will have to allow for the punishing effect of the pallet truck now being introduced, which exerts a point load exceeding 1½ ton and has small wheels.

Fig. 14 shows a typical cross-section of a large goods-shed floor recently constructed on the Western Region.

In locomotive sheds, blue-brick paving, formerly in general use and now sometimes omitted on economic grounds, had much to commend it since local surface damage could be made good by patching at less cost than cutting out and renewing plain concrete, which also tends to disintegrate when oil or cleansing fluids are split. Alongside some pits where connecting rods are removed, the Motive Power Department may demand a timber surface.

Solid floors also call for the provision of suitably sited pipe and cable ducts to which access chambers should be provided at intervals and particularly where the alignment of services changes direction. Well-designed cover plates to these chambers, which will interfere as little as possible with the uniformity of the floor surface, are essential.

In the ground floors of offices, with boarding laid on timber joists, openings for inspection should be provided in the dwarf walls supporting the joists, for dry rot may otherwise develop undetected.

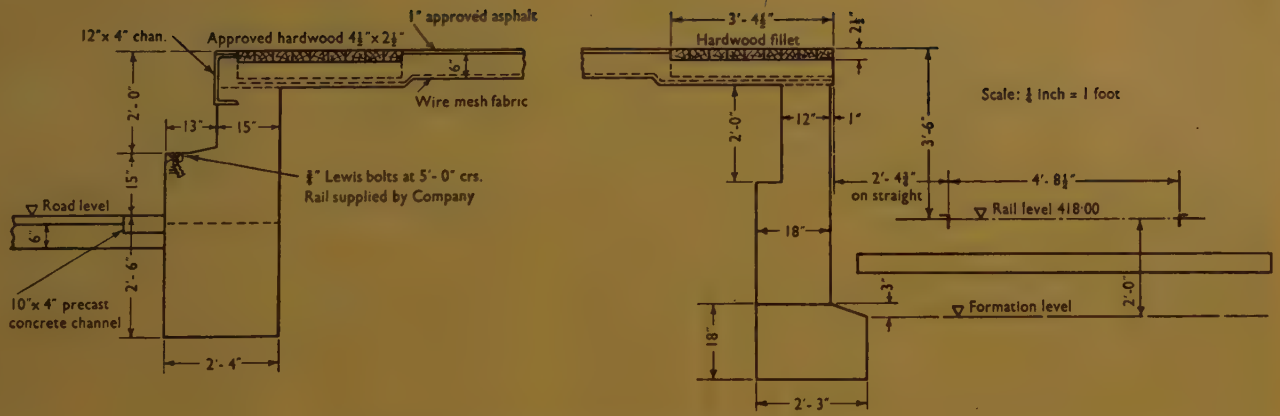
Walls of Buildings

Walls, in general, call for no special inspection facilities. The numerous types of wall in use in railway buildings cannot be fully covered in this Paper, but one of the principal problems with which the District Engineer has to contend, is the prevention and cure of dampness.

This may be caused by direct penetration of rain, damp rising from the ground, leakage from the roof or defective water services, penetration at door or window openings, or even atmospheric condensation.

There is no universal remedy and each case has to be dealt with on its merits. The resulting increase in maintenance expenditure can be very considerable, but properly designed cavity walls and correctly placed

Fig. 14



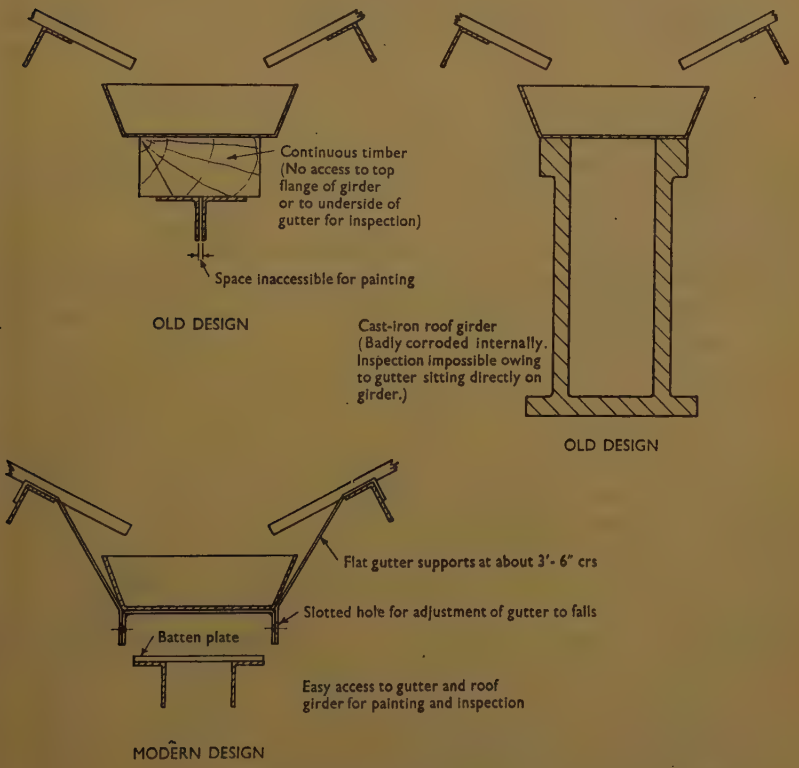
TYPICAL CROSS-SECTION OF A LARGE GOODS-SHED FLOOR

damp-courses, as prescribed in good building construction, will prove the best safeguard.

In exposed situations, external rendering is sometimes adopted to increase the resistance to the penetration of rain, especially where the main fabric is porous in character, and cement, lime, and sand mixes are most suitable. Rich mixes of cement and sand only, owing to their tendency to crack, are unsuitable.

Dampness in some of the older-type buildings, arising from faulty design, is beyond cure and the only steps the maintenance engineer can take to prevent further deterioration are the provision of external slating or tiling and internal linings secured to battens. These, in turn, may produce their own maintenance problems.

Figs 15


Continuous timber (No access to top flange of girder or to underside of gutter for inspection)
Space inaccessible for painting
OLD DESIGN
Cast-iron roof girder (Badly corroded internally. Inspection impossible owing to gutter sitting directly on girder.)
OLD DESIGN
Flat gutter supports at about 3'- 6" crs
Slotted hole for adjustment of gutter to falls
Batten plate
Easy access to gutter and roof girder for painting and inspection
MODERN DESIGN

VALLEY GUTTER DESIGNS

Gutters and Down-pipes

There is a tendency for designers to make valley gutters too narrow; these should be wide enough for a man to walk in without difficulty, since

they generally constitute the only access to roof valleys. For this reason, they should be of robust construction when not continuously supported; another reason for this requirement is that valley gutters are more difficult to renew than eaves gutters and leakage from them is likely to have more serious consequences.

When valley gutters are continuously supported, moisture makes its way between the underside of the gutter and the wall plate, setting up hidden corrosion, and this form of construction should be avoided wherever possible, because the underside of the gutter cannot be painted. (See *Figs 15.*)

Designers should avoid the use of purpose-made cast-iron gutters and down-pipes, especially the latter of square section built in flush, for, in addition to the difficulty of replacement, undetectable leakage from them may affect the main fabric.

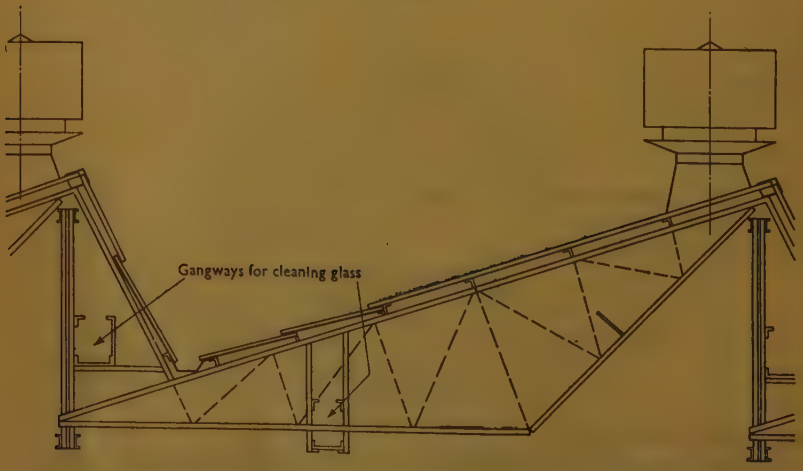
Cleaning eyes should be provided at all bends and at intervals in unduly long lengths.

Roofs

It is not possible within the scope of this Paper to discuss fully the numerous types of roof met with in railway structures, but their inspection and maintenance ranks high in the District Engineer's list of responsibilities.

With the possible exception of locomotive-shed roofs, there is a tendency for inspection to be restricted to the times of re-painting and, with the larger overall spans generally at some considerable height from the ground,

Fig. 16



WALKWAYS INCORPORATED IN THE DESIGN OF A NEW ENGINE-SHED

especially over lines carrying traffic, the provision of permanent inspection gangways would be a boon to the roof examiner without adding greatly to the first cost of the structure.

There is also a strong case for similar facilities in locomotive-shed roofs, owing to the necessity for more frequent inspection of roof trusses and attention to roof lights and smoke chutes. These are seldom provided, but *Fig. 16* shows a walkway incorporated in the design of a new locomotive shed on the Western Region. *Figs 17* show an inspection gantry used on the roof of Paddington Station.

The conventional umbrella or cantilever platform covering is usually low enough to be reached by ladders or mobile gantries.

There is some difference of opinion as to whether these coverings should slope inwards or outwards. With the outward sloping roof, if the eaves gutters require attention, they can only be reached from the platform line, entailing Sunday or night occupations at busy stations. With an inward sloping roof, nearly horizontal lengths of down-pipes are eliminated and the valley gutter can be reached more easily, but chokage by leaves or snow might have more serious consequences.

Whichever type of roof is adopted, rain-water down-pipes should never be sited within cast-iron columns or built-up stanchions.

In offices and other buildings with ceilings, trapdoors for access to the roof space should be provided for inspection purposes, but these should be kept locked to prevent the roof space being used as a lumber room and loads being placed on ceiling joists which are not designed to carry them.

The ends of mild-steel roof trusses supported on brick or masonry walls should not be built in but should be bolted down to pad stones with adequate space left for inspection and painting.

Where louvres are provided to permit the escape of, or to assist the extraction of, smoke and steam, these should be constructed so far as possible of non-corrodible material.

At one important station, the Author experienced considerable trouble with steel-plate louvres in the lantern of a large roof which, owing to severe corrosion, had to be removed pending replacement by plain asbestos sheet with non-corrodible fastenings.

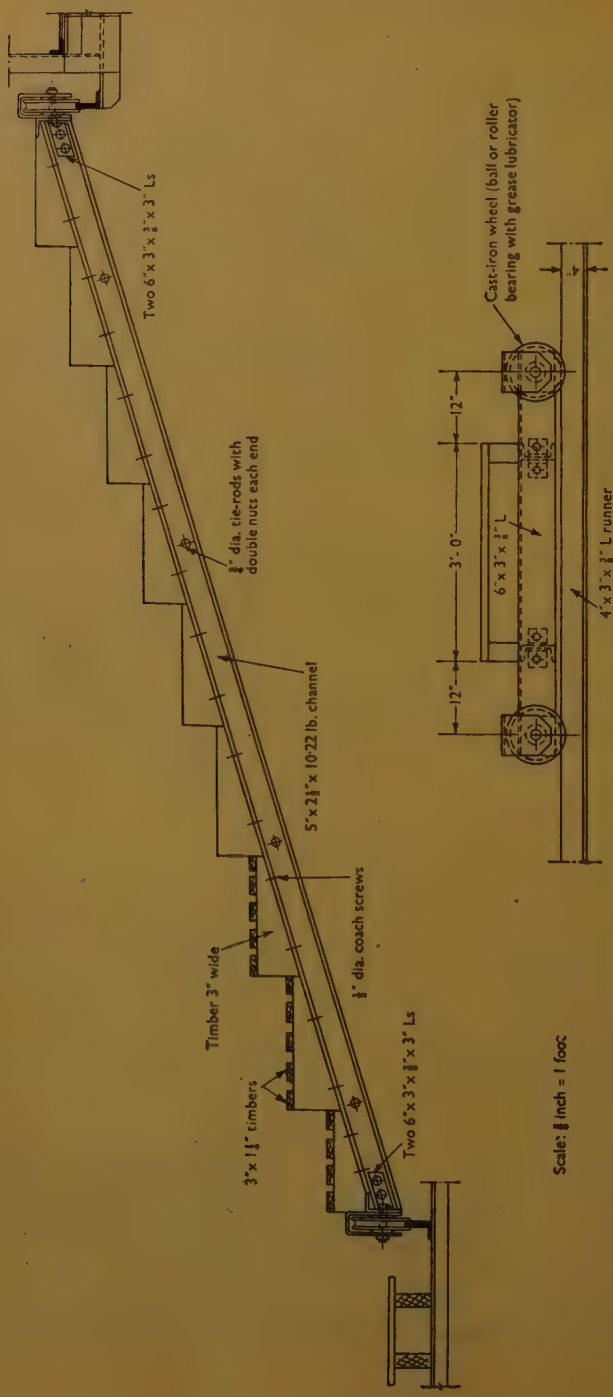
Where smoke extraction is necessary, directional ventilators such as louvres should be avoided, circular non-directional-type ventilators being more efficient.

Glazing Bars

Manufacturers, after many years of experience, have produced many different types of patent glazing bar, the majority consisting of a steel core in a non-ferrous metal sheath. These, in general, give satisfactory service and the designer may specify the type with which he is familiar with confidence.

The latest addition is the aluminium-alloy bar, which has also proved

Figs 17

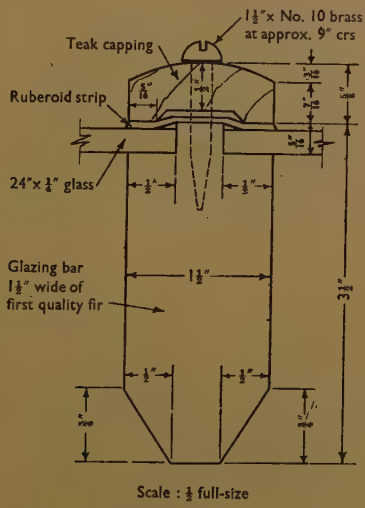

Two 6" x 3" x ⅜" x 3" Ls
1" dia. tie-rods with double nuts each end
5" x 2½" x 10·22 lb. channel
Timber 3" wide
1" dia. coach screws
3" x 1½" timbers
Two 6" x 3" x ⅜" x 3" Ls
Cast-iron wheel (ball or roller bearing with grease lubricator)
12"
3'-0"
12"
6" x 3" x ⅜" L
4" x 3" x ⅜" L runner
Scale: ⅛ inch = 1 foot

INSPECTION GANTRY USED ON THE ROOF OF PADDINGTON STATION

satisfactory, but the designer should acquaint himself with site conditions before selecting this bar, since it cannot be employed where strong acid has to be used for cleaning roof glazing.

The Author has found the standard G.W.R. timber glazing bar to be equal to the patent glazing bar, and a definite advantage in locomotive-shed roofs, since the capping can easily be removed and the glass taken out for cleaning. In smoky sheds, this obviates the necessity of the maintenance staff working in bad atmospheric conditions when the underside of the glass has to be cleaned. (*Fig. 18.*)

Fig. 18



STANDARD G.W.R. GLAZING BAR

Smoke Chutes and Stacks

A really efficient smoke chute that is economical in maintenance has so far eluded both the designer and the maintenance engineer, and the numerous experimental types which have been tried might well form the subject of a separate Paper.

It is perhaps not possible to achieve perfection with the varying conditions found in busy sheds and, inevitably, a considerable volume of smoke escapes, producing the worst possible atmospheric conditions for the roof structure.

The old-type timber smoke chute is fairly efficient but the corrosive effect of the atmosphere on hanging iron, nails, and other fastenings, and the consequent sagging of frames and loosening of boarding involves the District Engineer in constant expenditure.

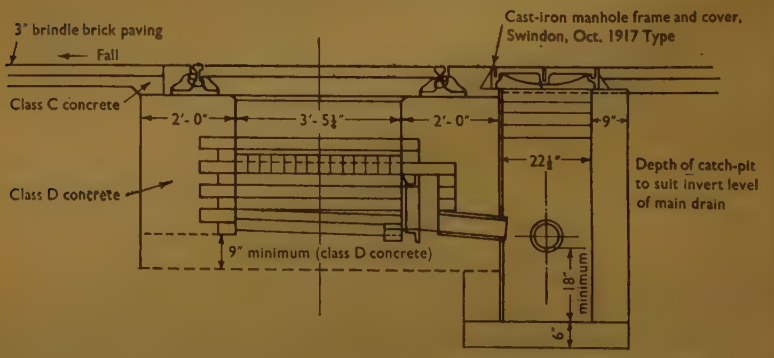
Some economy has been achieved on the Western Region by the use of asbestos cloth secured in frames before erection, in place of boarding. This stands well in the sides of chutes, but is liable to disintegrate on the tops if subjected to engine blast. (*Fig. 19*, facing p. 381.)

In one shed on the Author's district, purpose-made asbestos chutes have so far given no trouble, but the first cost is high and, in the event of damage, replacement of units is even more costly.

Engine Pits

Another frequent source of trouble, involving heavy maintenance expenditure, is the chokage of drains from pits at which boiler washing is carried out or where ashes are dropped. Although it is customary for gratings to be secured to prevent their being lifted by the shed staff, sludge, fine ashes, and waste find their way into the drains. At one busy depot, the Author had a main drain choked in this way for 6 months. The designer should, therefore, include in the pit lay-out intercepting chambers outside the pits with vertical gratings and sumps, which can be cleared periodically. An example of an ash interceptor which has proved satisfactory is shown in *Fig. 20*.

Fig. 20



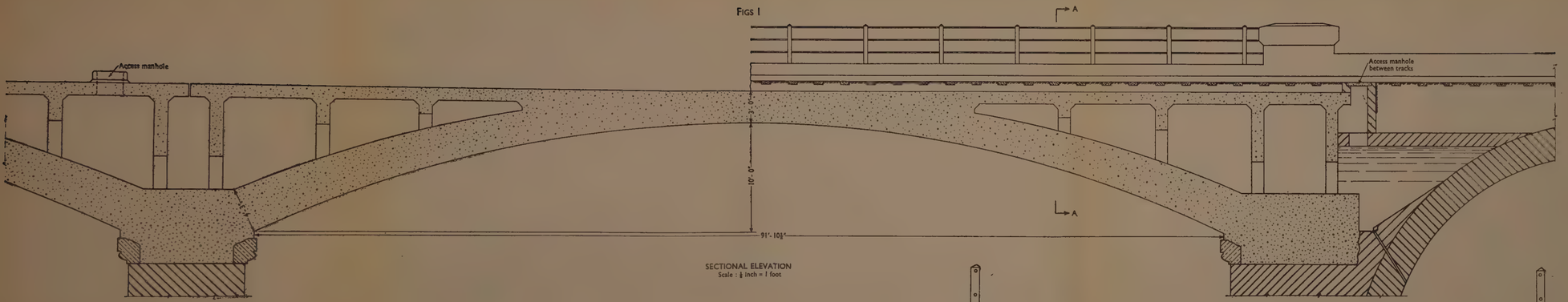
Satisfactory Design of Ash Interceptor

A new factor in engine-pit design has arisen with the introduction of engines from which fires are dropped without quenching.

This has had disastrous effects on pits with concrete walls and, unless this problem is tackled by the designer quickly, the resulting effect on pit maintenance will soon manifest itself wherever these engines are introduced. If the designer selects fire-clay or fire-resisting bricks to line his pit, he should ensure they are properly bonded into the side walls.

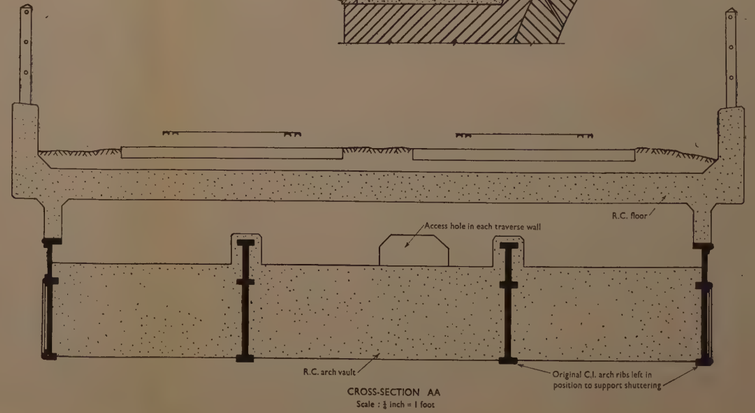
Pit lighting incorporated in modern engine-inspection pit design will

FIGS 1



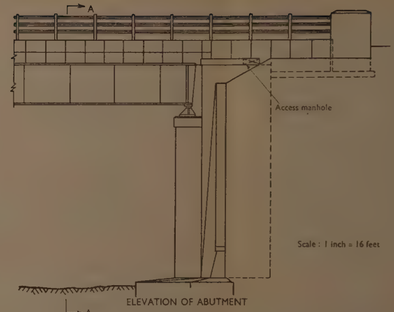
SECTIONAL ELEVATION
Scale : ½ inch = 1 foot

OPEN-SPANDREL-TYPE SHEET ARCH



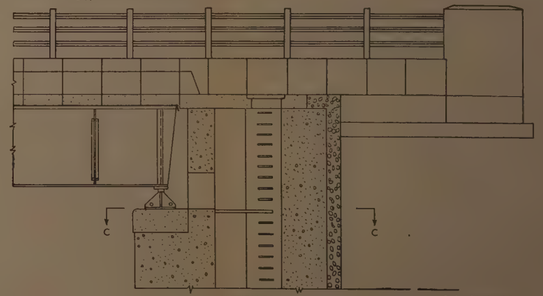
CROSS-SECTION AA
Scale : ½ inch = 1 foot

FIGS 2



Scale : 1 inch = 16 feet

ELEVATION OF ABUTMENT



SECTION BB

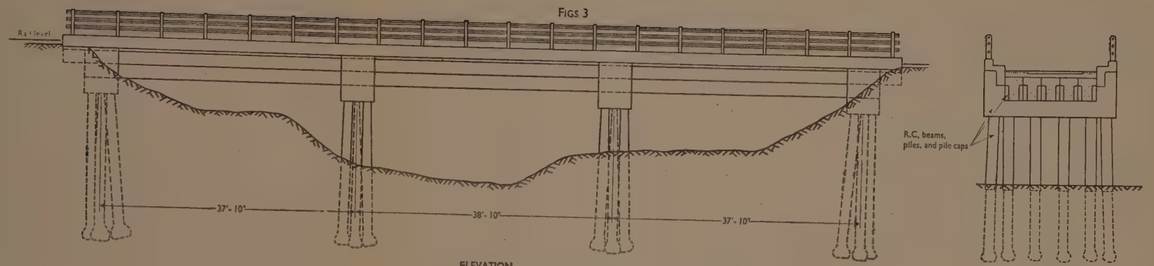
Scale : ½ inch = 1 foot

MASS ABUTMENTS AND WING WALLS

WILLIAM CLOWES & SONS, LIMITED : LONDON

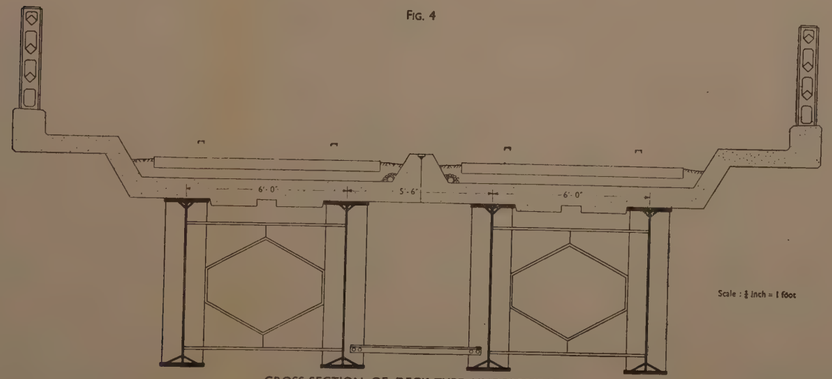
THE DESIGN OF STRUCTURES IN RELATION TO MAINTENANCE AND INSPECTION

FIGS 3



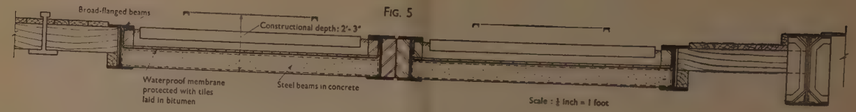
SUBSTRUCTURE DESIGN USING TRESTLE BENTS

FIG. 4



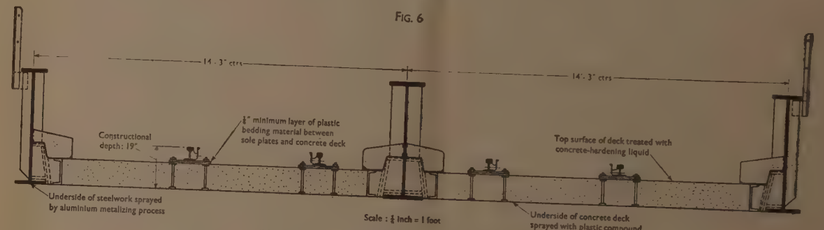
CROSS-SECTION OF DECK-TYPE UNDERBRIDGE

FIG. 5


Broad-flanged beams
Constructional depth: 2'-3"
Waterproof membrane protected with tiles laid in bitumen
Steel beams in concrete
Scale : ½ inch = 1 foot

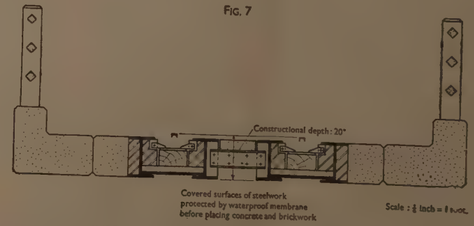
CROSS-SECTION OF SMALL-SPAN THROUGH-TYPE UNDERBRIDGE

FIG. 6


14'-3" ctrs
14'-3" ctrs
Constructional depth: 19".
½" minimum layer of plastic bedding material between sole plates and concrete deck
Top surface of deck treated with concrete-hardening liquid
Underside of steelwork sprayed by aluminium metalizing process
Scale : ½ inch = 1 foot
Underside of concrete deck sprayed with plastic compound

THREE-GIRDER UNDERBRIDGE, WITH RAILS ON CONTINUOUS PLATES BOLTED TO BRIDGE FLOOR

FIG. 7


Constructional depth: 20"
Covered surfaces of steelwork protected by waterproof membrane before placing concrete and brickwork
Scale : ½ inch = 1 foot

IMPROVED DESIGN USING TROUGH GIRDERS

FRANK TURTON and N. S. COX

eliminate the cutting of light-leads when engines are moved, besides providing better working conditions.

Conclusions

The designer, without serving a long apprenticeship on maintenance, cannot be expected to be fully aware of all the difficulties to be overcome by the maintenance engineer in inspecting and in maintaining the structures in his charge.

It is equally true to say that the maintenance engineer may be lacking in his appreciation of the designer's difficulties when required to design a structure at a figure that is economic in relation to its purpose.

Their several difficulties can be reconciled to a practical degree by close co-operation and an exchange of views during the periods of design, construction, and subsequent maintenance.

Acknowledgement

The Author tenders his thanks to Mr M. G. R. Smith, M.B.E., B.Sc., M.I.C.E., for facilities to present this Paper and for permission to make use of information, drawings, and photographs which are the property of the British Railways, Western Region.

The Paper is accompanied by three photographs and eighteen sheets of drawings, from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Discussion

The Chairman, when proposing the vote of thanks to the Authors, said that railway engineers in Great Britain felt that structures of all descriptions had a critical life of between 80 and 100 years, and unfortunately many of them were reaching the end of that life, so that, as Mr Turton had hinted, British Railways were facing a serious problem from the financial point of view. He could not say when it was going to be possible to work out the life of their bridges and depreciate them as they ought to be depreciated.

Mr Turton had been very wise to emphasize the need for the avoidance of cheese-paring in design. In the course of a long railway career, the Chairman had seen that fault for himself. The Authors had both emphasized the need for sufficient concrete cover to reinforcing bars; that was equally important and its neglect was causing a great deal of expense.

The Chairman agreed with Mr Cox that it was very desirable for the

maintenance engineer, whether a District Engineer or not, to be consulted about new designs, since he would have to maintain them. It was also very important that every part of a structure should be easily accessible for inspection and maintenance.

Mr E. K. Bridge agreed with Mr Turton that fine limits in theoretical design were not very practical, more especially since the data with regard to loading were not accurate within narrow limits. Fortunately the designer had not usually to concern himself with the stresses at points of local concentration, such as rivet holes, in view of the redistribution of stress which occurred the first time that that local stress exceeded the yield point.

The rising prices which had been referred to considerably affected the economics of bridge maintenance and renewal. In the past it had frequently been considered that, if a reconstruction could be deferred for about 15 years, the saving of interest on the cost of reconstruction would be equal to the cost of that reconstruction, but under present conditions the cost at the end of 15 years might well be more than double the present cost, so that little or no saving in interest on capital was effected by deferring renewal. When the increasing costs of maintaining the old structure were taken into account, it might well be found that the delay had been uneconomic.

It could easily be shown by a simple graph, comparing compound interest on capital with rising prices, that to defer work costing, say, £1,000, as had often to be done recently, owing to shortage of material, labour, and money, might mean that when it was renewed it was going to cost a great deal more than it would have cost if the work had been done earlier. He did not suggest that it was necessary to consider the wholesale renewal of bridges ; his point was that delay in a period of rising prices was a very serious matter so far as maintenance and renewal was concerned.

A point to be considered in connexion with design was the avoidance of contact between timber and steel. Where they had to be in contact, as in the case of old bridges which were only partially reconstructed, he would emphasize the value of putting some form of protection, such as hessian-based bituminous sheeting, on the steel (sealed on to the steelwork) to keep the wet timber off the steel. The number of cross-girders on bridges which had been destroyed by being notched-out where the main timbers rested on a short section of the flange was surprising, and where timber was renewed on an old bridge he suggested that some sheeting should be placed on top of the steel to keep the steel dry.

The Chairman and the Authors had mentioned cover for the reinforcement in concrete beams. Mr Bridge understood that a meter had been evolved which could measure the thickness of the concrete cover to the steel reinforcement in beams by some electro-magnetic method. That should provide a check, because it was not until they began to spall that the unsatisfactory nature of some beams was discovered.

Mr Turton had said that it was not desirable to use the bridge deck to strut the abutments. Mr Bridge agreed with him on new designs, but sometimes that could be very useful on old structures, where the abutments began to fail and tilt inwards, and a bracket on the underside of the girders could often make the bridge stable again and give it a much longer life.

Referring to Fig. 6, Plate 2, it would be interesting to know whether the Authors had actually carried out any such construction with the chairs so closely supported by the concrete, because under high impact one would expect shattering. It would also be of interest to know why Mr Turton considered a welded girder much more difficult to repair if it corroded than a riveted girder, bearing in mind the templeting and reaming necessary to make the rivet holes in the new plate match up exactly with those in the existing structure.

Mr P. S. A. Berridge said that if the contents of the Paper had been properly appreciated when the railways in Great Britain were built, the problem of maintaining so many worn-out and decrepit bridges and other structures would not be so formidable as it was today on British Railways. Many old bridges had been allowed to remain in the line because the permissible stresses for wrought iron and steel had been increased, a better understanding of load distribution, and so on, making it possible to reduce the "factor of ignorance." At the same time, however, the factor of safety was being considerably pared down, and he wondered whether that was wise, knowing that the bridges at the present time did not receive the attention which they ought to have.

The Authors blamed shortages of materials and labour for the increasing arrears in bridge reconstruction. Mr Berridge was of opinion that those shortages could be overcome, but mentioned another reason for the delay, namely, the time which was lost in obtaining authorization for worn-out bridges to be renewed. One could not help sympathizing with the local authority concerned with a road bridge who wanted to take advantage of the reconstruction to improve the alignment of the road, but there were many bridges which had been scheduled for reconstruction long before the last war, but where permission to get on with the reconstruction was still being withheld. Even when agreement had been reached with the local authorities and everybody else concerned, many months elapsed before the engineer was allowed to get on with the job. The policy of centralization which had followed nationalization had not helped matters.

Week-end occupations were liberal enough in Britain, and by careful planning and erection schemes it was possible to keep the length of time for which speed limitations had to be enforced down to the minimum. He was sure that when the time taken to obtain authorization was reduced, and the engineers were given a free hand to get on with the job, the arrears of reconstruction could very well be overtaken.

Turning to the all-important need to design with a view to future maintenance, he thought that there were four fundamental principles to

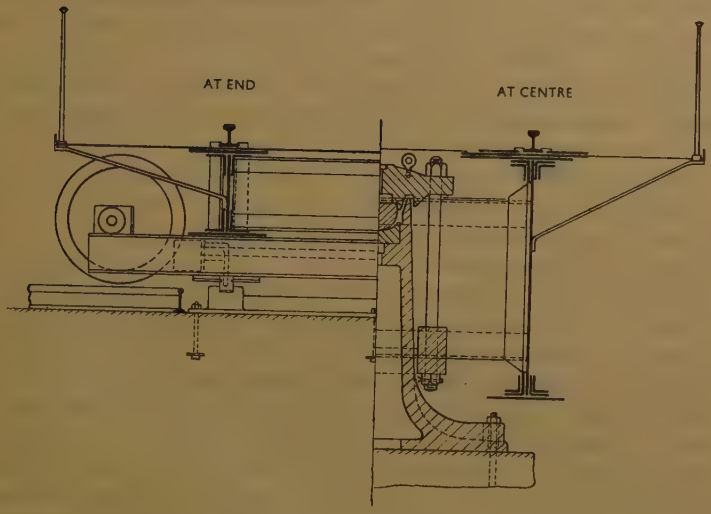
bear in mind in the design of girder work. The first was adequate strength in *all* parts. That might sound obvious, but it was not enough to design the main girders, the floor members, and so on, to take care of the principal stresses if the effects of floor interaction, lateral forces from nosing loads, and secondary and deformation stresses were neglected. Deformation stresses in a triangulated girder could well amount to 16 per cent of the principal stresses, and should not be neglected.

Secondly, freedom of movement had to be provided for wherever such movement would inevitably take place. In the past, far too little attention had been paid to provision for expansion and contraction, with the common result that, quite apart from the damage to the bedstones and masonry, the steelwork itself had been badly distorted. Mr Berridge showed a slide illustrating such a case; the top flange had been bent away, and the bearing below had dug into the bedstone. Mr Turton had wisely drawn attention to that sort of failure and had shown, in Figs 2, Plate 1, an excellent design of bearings for a deck-type bridge. In through bridges the decking should be supported only on the main girders and kept absolutely clear of the abutments. *Fig. 21*, facing p. 381, showed how the girderwork of a road bridge had been kept well away from a hillside which was prone to landslides. It would have cost a tremendous amount to provide permanent arrangements, but the light trestling which connected the end of the girderwork to the river bank could be swept away and it would not be very serious, because it could easily be replaced and the road restored.

Thirdly, Mr Berridge emphasized the need for accessibility for painting and for proper drainage and ventilation wherever steelwork was exposed. He referred to a locomotive turntable, which was probably one of the most ill-treated pieces of girderwork on the railways. Many turntables had originally been designed (see *Fig. 22*) with the rail laid directly on top of the girder, the flat-plate walkway decking supported on a very feeble bracket, which led to a sagging outwards, the outer edge-angle then being in a position to collect all the water and hold it. The main pivot in the centre of a balanced turntable had a very small area of bearing which invariably wore out very quickly, and there was a very small locking wedge. When an engine was being turned, the men, instead of trying to stop the table from turning by holding on to the levers, usually inserted the wedge, thereby imposing a tremendous load on the little locking pocket.

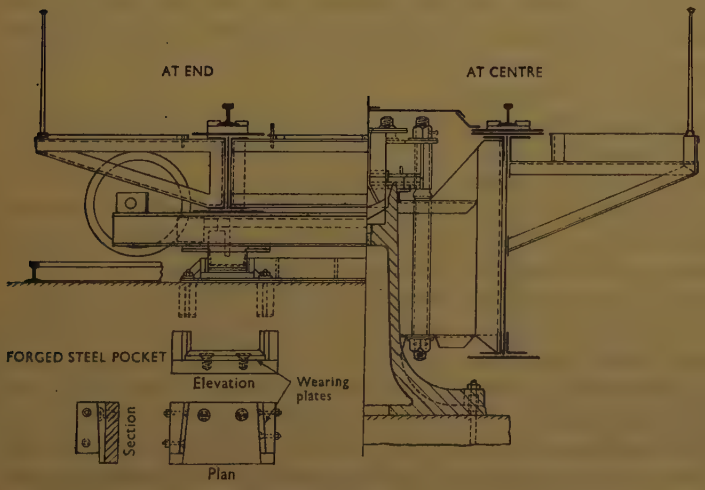
Figs 23 showed how such a turntable had been improved for ease of maintenance. The decking had been kept clear of the edge of the flange of the main girder, so that it was possible to paint all round. The deck plate between the main girders had been taken away and replaced by a grillage, allowing the passage of light and air and removing the "hothouse effect" obtained by enclosing everything underneath the girders. The edge-angle was turned down, so that there was no trap for water. A large

Fig. 22



LOCOMOTIVE TURNTABLE BEFORE MODIFICATION

Figs 23



TURNTABLE AFTER MODIFICATION

and efficient wedge-type locking arrangement was fully capable of taking the force of stopping the turntable if the men put the wedge into the locking pocket, and the centre pivot had a floating bronze bush of ample dimensions.

The fourth principle enumerated by Mr Berridge was proper attention to details. One example was the need for the very special attention which should be given to that most vulnerable part of steelwork, namely, where encased steel met wind and weather. Frequently, where a steel column had been taken down into the foundations, all the steel in the foundation had been wasted, because the column had rusted at the vulnerable spot where it went into the ground.

He showed a slide illustrating how the columns and steelwork of a foot-bridge staircase had all been kept above ground, so that where the most vulnerable point occurred the bolts were well protected by the baseplate, and the nut on the top of the holding-down bolts protected the bolts themselves. If it was necessary to develop a moment at the end of a column, it should be taken by the holding-down bolts, which could be designed to take it. If it was on a station platform and it was not desirable to have the base of the column standing proud of the platform level, it could be put in a recess and the recess drained out to the line at the side, a grillage being provided over the top which would keep the dirt out but which could be lifted up in order to inspect and maintain the bottom of the column.

Finally, Mr Berridge referred to the life of iron and steel. His last slide showed an iron pillar which had been erected in the 3rd century B.C., which had not yet been painted. It was in very good condition. It was true that it had not been subject to a corrosive atmosphere, but if girderwork were regularly painted and rust was not allowed to start, a properly designed bridge could have a very long life, and such girderwork could be taken apart and reassembled elsewhere if its life exceeded that of the railway.

The Chairman remarked apropos Mr Berridge's observations, that the Railway Executive had to have a broad back, and that it had been the target for a lot of unpleasant allegations quite recently. He could not, in any case, accept for one minute Mr Berridge's contention that centralization had had any effect on either accelerating or decelerating the rate at which bridge construction had been carried out.

Mr J. S. Campbell said that in the study of how design should take account of maintenance, a great deal of ground had been covered, not only in the present Paper but in other Papers which had been read recently before the Division. A stage had now been reached where the problem of designing with a view to keeping maintenance very much in the fore-front of the design was fully appreciated. He would not say that the designer at the drawing-board was not aware of the factors for which he had to cater when designing, nor would he suggest that the designer had

forgotten some of the factors which he had learnt earlier in his experience, but there were certain difficulties at the present time, particularly in so far as technical labour was concerned. The men who had been designing for a number of years were not getting any younger, and there was the question of who was to replace them. The usual source for recruiting technical labour was provided by the schools, colleges, and universities, and he was a little apprehensive about what the railways were going to do in 6 or 7 years' time when the older men had retired and the younger ones had come in to take their places. He did not want to belittle the younger people, nor did he say that they had not the interest in the problem which their predecessors had ; but, as had been pointed out by both Authors, it took some time to understand fully and put into practice all the relevant points, such as those dealt with in the Paper.

He thought it would be a sensible idea for the selected younger men to be allowed to go into the Districts for a period of $1\frac{1}{2}$ to 2 years,there to carry out the examination of structures, under a supervisor. That was probably the best way of finding out how not to design. They should do that not so as to be absorbed into the normal routine District work but so as to be given the opportunity, as men selected because of their bent for design, fully to apprehend the difficulties which they would try to overcome when they were back again at the design board.

It was clearly stated in the Paper that a high standard of efficiency in design could not be expected if insufficient attention was paid to matters of maintenance. It was mentioned that a clear indication should be given that bearings had to be accessible and that the ends of girders had to be clear in order that proper inspection could be undertaken. Details at stiffeners had to be accessible. He hoped that within the next 10 years sufficient research evidence would be available to enable designers to dispense with stiffeners, at least on the outside of girders, because if there was anything which caught the moisture and created corrosion it was stiffeners and their details. Simplicity was the real policy for design, and he would like to instance the value of straightforward prestressed concrete slab or beam construction wherever it could be used.

On the reconstruction of three-girder through-type bridges it was worth while to alter them to two single-track bridges. Mr Turton had stated that, but Mr Campbell would like to go further. Even if it meant the lengthening of abutments and even the possibility of having to get a little more land for the purpose, he thought it would pay in the long run. Many engineers had had the experience of dealing with the reconstruction of bridges using cross-girders encased in concrete and had seen the traffic going over one half of the bridge, thus causing a bending effect right through to the cross-girders of the uncompleted half ; as a result there was distortion or an increasing of the gap between the concrete and the joist until there was considerable uncertainty as to what use the concrete was as encasement in those circumstances.

Fig. 4, Plate 2, showed a very simple construction of the type which had been used in the replacement of the Scottish bridges damaged by flood in 1949. It would be noted that there was perhaps only one place which was not accessible, and that was where the well decks rested on the welded girders, but that could be met, and any reasonable fear of corrosion there overcome, by the introduction of a bitumastic felt.

Fig. 6, Plate 2, showed a composite construction of a type recommended by the Bridge Sub-Committee. The difference was that it carried a non-ballasted track, and the track was bolted direct on to the prestressed concrete deck. His only qualm was that he would like to know in time how the bolts behaved. The bolts went through cored holes and, owing to longitudinal and transverse forces, there was just a possibility of the holes becoming elongated. He did not know if any serious damage would result. It might be possible to grout afterwards, but he wanted to know how the structure in question behaved.

Referring to *Figs 11*, Mr Campbell agreed with Mr Cox that the gap between the angles of rafters and tension members had always been troublesome, but he thought that the suggestion which was made in the lower part of the Figure was not a good one. Instead of that type of design, he would rather use a tee, or if that was not possible, a broad-flange beam cut for the purpose. The same would apply to *Fig. 16*, which showed a north-light roof. He thought that a north-light roof was difficult from the maintenance point of view. Instead of the vertical transverse girder, which was shown there to take the total load, he would prefer to see that the girder was inclined to the angle shown for the glazing framing, and instead of a framing such as took the glazing one would then have an inclined long girder at the required angle, which would carry the load and, at the same time, carry the glazing, with stiff joints top and bottom; that, he thought, would be a much more economical structure and much more easily maintained.

Mr J. D. West observed that the main theme of the Paper was the importance of maintenance. In that respect he was in full accord with Mr Turton.

In illustration of the principles of design set out in the Paper, he described the failure and repair of the abutment of a bridge on the London–Ipswich–Norwich line, carrying the line over the River Stour at Manningtree. It was of a comparatively modern design but the abutments, as shown in *Fig. 24*, has disintegrated vertically and laterally, and a speed restriction had had to be imposed. That had been an embarrassment to the Traffic Department, because the bridge was situated at the bottom of a gradient, and speed at that point was an important factor in climbing the gradient. As an immediate measure the timbering shown was put in, and it had been designed quickly in conjunction with the permanent measures.

Fig. 25 showed the permanent measures, which included what was

virtually a new abutment in front of the existing one, carried on bored piles, which also gave lateral support to the old abutment. The work had been so arranged that it could be undertaken between the temporary timbering, and a good deal of it had been done without a very serious speed restriction on traffic.

Fig. 24

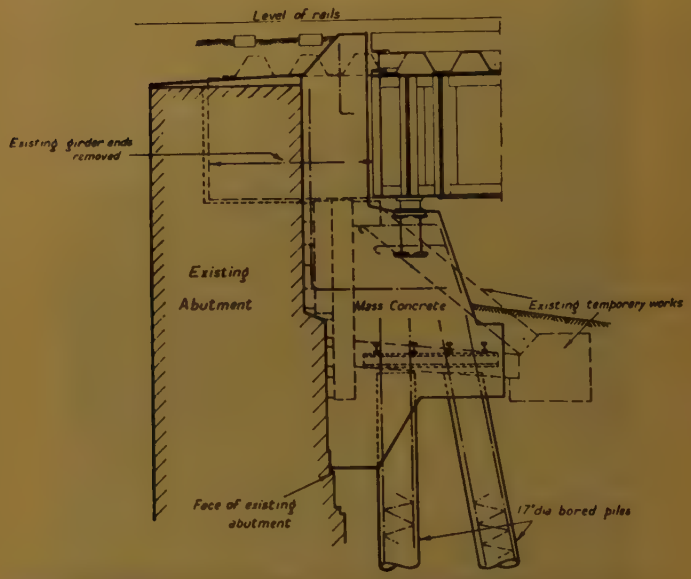
FAILURE OF ABUTMENT OF BRIDGE CARRYING THE LONDON–IPSWICH–NORWICH LINE OVER THE RIVER STOUR AT MANNINGTREE. TEMPORARY TIMBERING IS IN POSITION

The temporary timbering was completed by the District Engineer and the site handed over to the contractor to carry out the permanent repairs. The opportunity had then been taken to shorten the girders and set them on pedestal bearings, introducing a web stiffener over each bearing. Up-to-date practice had been applied further to the repair to make the girder ends completely accessible for maintenance ; it was an example of some-

thing having happened and of something being done quickly to restore traffic facilities, followed up by permanent repair work to modern conceptions of design and undertaken without undue disturbance to the temporary work which had first been erected. *Fig. 26* showed the finished repair.

One of the bridges carrying the Newark–King's Lynn trunk road was a combined road and rail bridge with a swing span. Some time ago it had become necessary to re-deck the road portion because of the decayed

Fig. 25



PERMANENT MEASURES FOR REPAIR OF ABUTMENT SHOWN IN *Fig. 24*

condition of the timber. The work had been carried out by employing a battle-deck type of floor, of plates with ribs welded on the underside. The decking had been prefabricated in sections to the camber of the road, and a 1½-inch layer of asphalt laid on the top. Inverted transverse tees had been used to break up the area of the asphalt to avoid failure from expansion; they were also useful as a gauge of the correct depth of the asphalt. From the maintenance point of view that work had come at an opportune moment, because Mr West had felt that the timber deck had been on the bridge long enough. The corrosive influence of the timber where it made contact with the cross-girders had been beginning to make itself felt. The continuously welded deck was 100-per-cent watertight.

Mr West then referred to the encasement of steel with concrete and emphasized the value of thorough waterproofing just prior to the concrete

protection being applied. Throating was, in his view, a very valuable asset, because it prevented corrosion taking place "between wind and water" on the line between the steel and the concrete. In favouring protection of steel by concrete he was at variance with Mr Turton. It could, however, be very good if the reinforcement was put in with care and not as a mere token. He called to mind a bridge over a goods yard where there was a great deal of smoke. The webs of the girders corroded

Fig. 26

COMPLETED REPAIR TO THE ABUTMENT SHOWN IN *Fig. 24*

into very big holes, but the bottom flanges of those girders had been protected with concrete which had been there for 40 to 50 years. It had had to be hacked off to carry out a repair and the flanges had been found to be in perfect condition.

Mr M. A. Henry referred to the concrete warehouse illustrated in *Fig. 8* and described on p. 379. The illustration gave a good impression of the dilapidations, the repair of which was in hand. It was a typical example of a medium-size warehouse, consisting of a platform floor with three floors over, each floor being of about 10,000 square feet. It was a composite structure, the main fabric being steelwork with all external faces encased in concrete. The panels were $2\frac{1}{2}$ inches thick, reinforced with Hyrib. As would be seen from the illustration, each panel carried

three steel-framed windows. The floors were of hollow-cast construction, designed to carry loads of about 1½ cwt per square foot.

The warehouse had been completed about 1930. At that time there had been a great demand on the Great Western Railway for premises of maximum floor space and entailing the minimum of expenditure. The endeavour to economize on the original cost had been achieved in part at the expense of the panel design. It should be physically possible to construct panels 2½ inches thick and to make them watertight, but the inclusion of vertical framing to support the windows had introduced maintenance problems which, over the years, had proved very difficult to deal with. The warehouse was in an exposed position, and shortly after its completion wind and rain had started penetrating through the panels in the vicinity of the windows. Corrosion of the frames had set in, and that had started cracking and displacement of panes. The corrosion had spread to the light Hyrib reinforcement and cracks had formed. For the past 20 years it had meant a battle with the South Wales elements to keep the warehouse in a usable condition.

Deterioration of the window frames and panels had become so excessive that the warehouse floor space adjacent to all sides of the warehouse had become unusable. Eventually it had been decided to undertake major remedial repairs. Three schemes had been examined: (1) to remove the whole of the concrete casing and encase the whole structure in brickwork, including the building of brick panels; (2) to break out the panels and substitute "Big Six" asbestos sheeting supported on steel framing; and (3) to remove all the panels and defective concrete and reinstate by "cement gun" process. Method (1) was felt to be too expensive, and (2) to involve the use of a great deal of steelwork, and so (3) had been adopted, and a contract had been awarded to the Cement Gun Company and the work put in hand in July 1952.

Tubular scaffolding had been erected in sections to carry screens for the demolition of the old panels and to prevent falling debris from damaging the covered verandahs around the warehouse. The guniting work had been carried out from swinging cradles suspended from tubular cantilevers fixed at roof level. The cradles were raised and lowered on a $\frac{5}{16}$-inch stranded steel flexible rope and made fast to a curved safety hook attached to the top runners. The cradles were raised by a hand winding gear and lowered by means of a hand-operated spring-ratchet release. The cradles, each of which weighed 2½ cwt and accommodated two men, were 6 feet by 2 feet, and the height to the safety rail was 4 feet, thereby giving the men in the cradles ample confidence that they could work without risk of accident.

The demolition of the panels had been carried out by using pneumatic hammers, and had been undertaken in sections, without any interference with the normal business of the shed. As panels were removed, large canvas screens had been erected inside the building from floor to ceiling

to prevent dust and to protect foodstuffs in the warehouse from inclement weather conditions.

The angle framing in the panels had been rearranged to carry windows of a new type. The new 4-inch mesh electrically spot-welded B.R.C. fabric reinforcement had been secured with rustless steel wire and laced with additional strips of spot-welded mesh around the angle framing. The formwork, comprising 4-inch-by-1¾-inch timber uprights, spaced at 18-inch centres, had been lined with hardboard and faced with sheet metal. It had been made up in convenient units and was interchangeable throughout the job. It had been wedged and shored from the inside after the mesh reinforcement was in position.

The material used for guniting was 3 parts of coarse sharp sand and 1 part of Portland cement. It had been applied by spraying or shooting with a cement gun in two 1-inch layers on consecutive days, making a panel thickness of 2 inches. The formwork had been removed on the third day and the external faces had subsequently been cleaned down and brushed to form the finished surface.

In carrying out a guniting job of that nature, the nozzle of the gun was kept 12 to 18 inches away from the face of the work. There was what was known in the trade as "rebound," which referred to the separation of the larger sand particles from the general mass. In carrying out the work, those particles fell away to the screens below and left pockets of porous work which, after removal of the formwork, had to be hacked out, cleaned, brushed, and flash-coated from the inside.

The window frames, previously assembled, had been fixed in position from the swinging cradles and subsequently glazed, using ferromastic putty, which appeared to give satisfactory results. The whole of the exterior surfaces had been given a coat of Cemprover to safeguard against the penetration of dampness, and subsequently treated with a sprayed coat of Cullamix by means of a spray-can funnel worked off the main compressor.

The plant comprised a 75-horse-power Broomwade diesel-engined compressor supplying air for the pneumatic hammers and the cement gun, working at a mimumum pressure of 80 lb. per square inch, the gunite being delivered at a pressure of about 40 lb. per square inch. The maximum length of delivery hose used was about 100 yards and worked satisfactorily. A portable welding plant and 12-horse-power petrol-engined generator had been used for all welding of the rearranged angle framing and B.R.C. mesh.

Mr Henry concluded by showing a number of slides illustrating the work described and the way in which interference with the ordinary business conducted in the warehouse had been avoided.

Mr H. Shirley Smith said that one of the Authors had referred to corrosion in members made up of two angles back to back with a space between them. Mr Shirley Smith had seen a case of that only a few

weeks previously in an old roadway bridge over water. It had a large number of wrought-iron lattice cross-girders of such construction below the deck, and almost without exception the corrosion had been so severe that the angles had buckled outwards just clear of the gusset and had wasted away to the thickness of a sardine tin. The girders had virtually no strength left. A single angle or tee, or some similar section should, of course, be used in such circumstances.

He had very strong views on the concrete encasement of structural steelwork. He was wholly opposed to it, and he was glad to find that the Authors were. He had some sympathy with Mr West who had described an instance where the lower flange only had been encased, and he could quite see that in such highly exceptional circumstances there might be something to be said for it; but to encase a whole plate-girder was a dreadful expedient. If one were so wedded to concrete that one must use it, one should use it with reinforcing bars, but not use structural steel and then hide it away with concrete so that it was not possible to get at the steel to keep it in order; it was very doubtful that the concrete would do so.

Mr Turton had referred on p. 375 to the reliability of the properties of steel. It was pleasing to find that in their references to materials the Authors had not been panicked by any temporary shortages which they had encountered. Mr Shirley Smith had stated [1] in 1952 that the steel allocated to structural work in Great Britain was only a very small percentage * of the total output, and it took only a minute variation in supply and demand to turn a structural shortage into a surplus. He had said at the same time that if things went on as they seemed to be going there would be no shortage of steel towards the end of 1953. Some people had gone in for all kinds of other material—such as timber, concrete, and even aluminium. There was no need for Mr Shirley Smith to repeat his views on them. The obvious material and the best and most practicable to use for bridges was steel, and all that had to be done was to find a proper way of protecting it. Previous speakers had shown examples of the use of steel in common-sense ways that made it easily accessible for protection and maintenance.

The Authors had referred to welded and riveted structures, and preferred welding. He believed, as probably most engineers believed, that welding would ultimately completely supersede riveting, but care had to be taken not to go too fast. Some curious failures had occurred in welded structures, and it was obvious that much had still to be learned. Recently there had been the failure of the all-welded Duplessis bridge in Canada. That was a bridge of heavy plate-girders, situated in a very cold country, which had collapsed unaccountably and smashed its way through several

[1] John Ratter *et al.*, "Relative Economics of Pre-stressed Concrete compared with other forms of Construction—Aluminium; Reinforced Concrete; Steel; and Timber." Proc. Instn Civ. Engrs, Pt I, vol. 1, p. 390 (July 1952).

* The actual figure was about 5 per cent.

feet of ice. After the wreckage had been extricated, Mr Howard, the consulting engineer, had said that none of the welds had failed. It was thought that the collapse of the bridge might have been caused by the effect of very low temperatures on steelwork of varying thickness which had different degrees of protection. There was a good deal of talk today about brittle fracture at low temperatures, and he believed it was a problem now under investigation.

Other matters requiring further study included the effect of over-stiffness, notches, concentrations of welded joints, abrupt changes in cross-section, and the specification of the steel which was being used.

Examples of extremist views on welding *vis-à-vis* riveting were to be found in New South Wales. There the Department of Main Roads had adopted welding enthusiastically and would not use riveting at all if they could help it. On the other hand, the Department of Railways, who were much more conservative, remained faithful to riveting. In Auckland—which, though not a great industrial city like Sydney, might be expected to have certain resources—welding was so much the vogue that if a ship needed riveting repairs there was the greatest difficulty in finding a single riveting squad in the whole of Auckland. In Mr Shirley Smith's opinion the wisest course was to take the middle path and keep on with some riveted work, particularly in those situations where the welding problems did not appear to have been fully solved.

Mr A. H. Toms agreed with Mr Berridge's views on some questions which the design of larger structures brought very much to the fore in a way that the design of smaller structures did not, though the points were of equal importance in both if subsequent maintenance difficulties were to be minimized. For instance, when any type of member or connexion was specified in the design drawing, full consideration should be given to what primary or secondary stresses or strains it would have to sustain, and they should be provided for either by adequate strength or by sufficient flexibility to avoid over-stress or undesirable working of joints. That was where welding was doing a useful job, because, in view of the rigidity of connexions which was enforced, more attention had to be given to that matter.

With regard to concrete casing, he had not much experience of failures, but in 1950, when the Railway Mission was in the United States, they had been informed by the Chief Engineer of one railroad at least that concrete encasement had been abandoned because of large-scale failures.

It was interesting to find one of the Authors referring to the inherent strength of brick arches. Mr Toms had previously described [1] repairs to a brick arch viaduct about 70 feet high after one pier and two arches had been knocked out by a bomb. The calculations which he had made

[1] A. H. Toms, "Repairs to Railway Viaduct over London Road, Brighton, after Damage by Enemy Action in May 1943." J. Instn Civ. Engrs, vol. 24, p. 353 (Oct. 1945).

showed clearly that even with deadweight only it would have collapsed like a pack of cards if there had not been very high tensile strength in the brickwork. After tying the innermost pairs of piers together with steel rods to share the unbalanced arch thrust, he had been able to put a 10-ton crane on one side, without collapse of the bridge, though very slow creep movements had continued to occur. Those movements had been under close observation, but, to avoid any unnecessary " scare," the men working on the job had not been informed about them.

He had been glad to hear Mr Berridge point out something which had given a good deal of trouble, namely, the drag effect of flat bearings when a span shrank in the winter, causing cracking of the abutment or other damage. The Germans used very elaborate bearings. There was a bridge at Brunswick which had roller bearings for expansion transversely and longitudinally, one above the other. He did not suggest going to that extreme, but in the past more attention might well have been given to those points.

He was also of the view that it was quite wrong to expose the main stress-carrying member of a bridge, whether steel or reinforced concrete, to the direct blast of locomotives. He thought that a smoke-plate of some plastic material, or a vitreous-enamel-coated plate, should be provided which was renewable. He felt sure that that would be a long-term economy. The Eastern Region were to be praised for their pioneer use of super-sulphate cement, which laboratory tests by the Research Department had proved to have a far higher acid resistance than any other cement used for normal construction. Mr Toms had suggested its use in asbestos sheeting, but could not interest any firm in making a trial quantity. The Eastern Region had induced a firm to do so, and he believed that tests on those sheets showed excellent results.

A number of inquiries had been received about what to use for the construction of locomotive ashpits. It appeared that one District Engineer at least, who had used Adamantine bricks with a so-called refractory cement jointing, had had trouble with the jointing mortar, and it had been suggested by the Research Department that here again super-sulphate cement, though not absolutely ideal for mortar, would probably produce a far better job.

Another thing which had given considerable cause for inquiry was the use of acid cleaners on roof glazing. At the moment the problem had not been solved, but there seemed to be an indication that when designing a roof some thought should be given to what would be used to clean it.

With regard to smoke chutes in locomotive sheds, why have them at all? The Southern Region had designed a locomotive-shed roof (after very crude model experiments on the drawing-office bench with an electric fan) which had no smoke chutes, but just a flat cover over an A-type apex which gave good smoke extraction, and everything was easy to get at for

maintenance. The main girders were protected against blast by plastic sheets, which at the moment were behaving satisfactorily.

Another trouble which had been experienced was with rigidity under a rail. He had been surprised to see in the Paper the detailed design of a bridge in which the rails were put straight on top of the concrete floor slab with only a thin layer of plastic material—probably Plasoleum—in between. That, unless it had considerable resilience, might cause trouble, and unless the bolts were in resilient ferrules, with corresponding resilient washers under the steel washers, it did not appear to him that that construction would last very long if subjected to heavy and fast traffic. The S.N.C.F. (Société Nationale des Chemins de fer Français) had pioneered experiments with perforated rubber pads under rails, and he would be interested to know whether those rubber pads had been considered in the case in question, with corresponding resilient fastenings.

Mr Toms referred to suggestions that promising junior technical staff should be given 2 years' training under a bridge examiner, and a further remark by a subsequent speaker which gave the impression that he was not very confident of the ability of the younger generation to carry on the design work of those who would soon retire. Mr Toms felt very strongly that too much emphasis was often placed on lack of practical experience of the young engineer.

Constant reiteration of the theme that a young graduate was of little use until he had had a good deal of experience in a job could easily lead to feelings of frustration and despair.

He thought that young men trained in an English university, where the professors had themselves been through the mill and had done, and might even still be doing, practical work, should at least have covered the rudiments of design and should know something about the maintenance aspects of problems which would confront them when they came to design structures.

His personal experience was that such a university training fitted a man to engage at once in certain aspects at least of design which no amount of experience, without such training, could.

Mr Toms was in no way belittling the value of the right kind of practical experience which was so evidently of the greatest importance. He did feel, however, that some of the younger members present might not have taken very kindly to some of the remarks to which he referred, and on account of which he had felt prompted to speak.

*** **Mr F. P. Dath** recalled an occasion some years previously when he had suggested the use of welding for structures, only to be met with the remark, "We have put in rivets for the past seventy years," with the obvious inference. Whilst appreciating the caution necessary, he felt that the approach to new methods of construction had been too conservative

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*** This and the following contribution were submitted in writing upon the closure of theoral discussion.—Sec. I.C.E.

on some railways. Nevertheless he agreed with Mr Shirley Smith that welding could be suitably applied to some structures whilst riveting would be more practicable in others.

He agreed in some measure with Mr Tom's remarks, because railway experience appeared to him to be measured in decades. Anything less was dismissed as " five minutes." He recalled being in one bridge-design office where he had seen a series of calculations made by one of the designers for obtaining a bending moment. That gentleman had added up the result and then proceeded to evaluate the stress, which erred on the side of safety by about 400 per cent ; the only other thing he seemed to require was a little seasoning to keep it fresh ! It was obvious that he knew only the simple formulae and could not analyse the moments and stresses from fundamental principles. However, it could not be denied that theory without practice and experience was valueless, and Mr Dath made it clear that, in spite of his criticism, he was grateful for the experience he had gained in railway offices in his younger days.

Fig. 24 was a very good example of inexperience and lack of knowledge, because quite apart from expansion, which was an elementary consideration, any structure subjected to movement by alternating loading would suffer in the same way if restrained and the material was not suitable to withstand the induced stress.

The foundations of transmission pylons had necessarily to be below ground level, but no trouble arose from corrosion if they were properly encased in concrete brought up to about 6 inches above ground level. There had been failures where that had not been done and the corrosion had been proved to be electrolytic.

Mr Bridge had mentioned the disparity between the design stresses and those obtained under test. Mr Dath was interested in transmission pylons, and when a new type was designed it was subjected to various tests and finally tested to destruction before others were fabricated. Failure had ranged to within about 98 to 105 per cent of the ultimate load and that, he submitted, was proof of good design. He could not think of a more complex system of loading than that which obtained when broken conductors caused unbalanced loads.

A meter had been evolved to measure the deposition of galvanizing or paint on steelwork and that instrument could no doubt be suitably adapted to measure the concrete cover on steel.

Mr O. A. Kerensky agreed with Mr Turton's statement on p. 370 that " the designer's ability and skill should be tempered with an appreciation of the possibility of changing conditions," but did not subscribe to the methods proposed to express that appreciation, which was " adding a bit for luck " (p. 371).

Actual dead loads should be properly calculated, making allowance for a heavier form of decking only if there was any reason to think that such a future change was likely or desirable. Live loads should be specified

by the highest appropriate authorities with full provision for all foreseeable future increases. After the total maximum loads had thus been determined, the designer should treat them as such and he could not be permitted to exercise individual judgement regarding any further future increases. Only chaos would result from such a procedure. The designer's job was to design tightly and economically by the best methods available at the time.

In metal structures the provision of extra thickness of material over and above that required for reasons of strength and robustness was hardly ever justified. Even an additional $\frac{1}{8}$ inch all round would not prevent a badly maintained structure from disintegrating, whilst a well-designed and properly protected and maintained structure would last indefinitely, even if made from thin metal sheets. Money would be better spent on good details and on proper initial cleaning, followed by the best available protective treatment. Nothing could be more dangerous than an idea that a structure "had something to spare" and therefore could rust a little without danger. Once the rusting had set in and the steel surfaces had become pitted, ordinary methods of cleaning and painting were no longer applicable, since rust in the pitted portion could not be removed by any form of brushing and only expensive grit blasting or flame cleaning could successfully stop the decay.

The designer should follow three basic rules :—

(1) He should estimate maximum possible loads, having in mind future development.
(2) He should produce the most economic structure to carry those loads, taking into account the cost of future maintenance.
(3) He should make sure that all parts were either permanently protected from all harmful influences or were accessible for inspection and maintenance.

Mr Turton, in reply, said that Mr Bridge had mentioned that timber in contact with steel was not very desirable, and that where it could not be avoided the steel should be protected by sheeting. Mr Turton agreed with that, but did not think that a modern designer would use timber in contact with steel in a new bridge. It was necessary to have regard to the question of permanence, which was mentioned in the Paper. It seemed to him that any bridge designed nowadays had to be regarded as permanent, unless it was purely for a temporary purpose—for a period of 10 years or so—and a bridge with any timber in it at all could not be looked upon as permanent.

Various speakers had criticized Fig. 6, Plate 2, and Mr Bridge had asked if anything like it had been built. One was in course of construction, but it should be made clear that that drawing had been included in the Paper merely to show what could be done in reducing the construction depth of a bridge. It was not advocated that it should be used in every situation. The particular bridge in question was an intersection bridge

carrying approach lines to a goods warehouse. The end of the bridge, which had three spans, was only 50–100 feet away from the entrance to the goods shed, so that it was not going to be subjected to the dynamic action of high-speed locomotives. Mr Turton did not think that in any circumstances he would put in such a bridge under a main line. There had been an experience of that kind on the London Midland Region a few years previously, and it had not been a very happy one ; the bridge had had to be taken out. There the bridge had been supporting main line traffic, and it had pounded very heavily.

Mr Campbell's point about bolts becoming loose was one which could only be answered by experience. It was not expected that they would work loose, but no definite reply could be given until some experience had been obtained. Mr Turton did not think that they were detrimental to the actual slab ; that had been taken care of in the design.

Mr Toms was correct in suggesting that Plasoleum had been used in the case to which he referred. Mr Toms had asked whether rubber pads similar to those used recently on the S.N.C.F. had been considered. They had not, because little had been known of them. Their use would be well worth consideration, even at the present stage, because the bridge in question was not yet built.

Mr Berridge had suggested that the Authors blamed shortages of materials and labour for arrears in renewals. Mr Turton did not think that that was the case ; what he himself blamed, at any rate, was the physical impossibility of being able to carry out the work. If one looked at the rate of reconstruction since the beginning of the present century, the number of bridges which had been reconstructed in comparison with the total number on British railways was extremely small, and even if the rate were doubled it would not be possible to reconstruct all the bridges within any reasonable period of time. If it were possible to release the flood of labour and materials necessary to reconstruct them now, it would almost bring the railways to a standstill.

On the subject of secondary stresses, he agreed with Mr Berridge that they had to be taken into account, particularly in triangulated girders. Those were no longer common but the question was also important with plate girders—particularly the welded types which were now being developed. So far as the movement of bearings was concerned, Mr Turton could not think of anything more important in a bridge than that they should be absolutely free.

When preparing the Paper he had had in mind the column of iron which Mr Berridge had mentioned, and had thought that if it were possible to have in Great Britain the kind of atmosphere which prevailed where that column was, there would be no need to worry much about metal bridges. The important point, however, was that which Mr Berridge himself had mentioned, namely that there was no reason why, even in Britain, bridges should not remain for ever if rust was not allowed to develop. The

difficulty was to prevent rust from developing, but up to now Mr Turton had seen nothing which would do that.

He agreed with Mr Campbell, in spite of what Mr Toms had said, that the young men who were coming in should have practical experience. Mr Toms had claimed that young men with university training were fitted to engage at once in certain aspects of design. That might be the case, but one did not come in contact with all types of bridge in a short time, and there were many types ; before a man could claim that he had a grasp of what a designer should take care of from the point of view of maintenance and inspection he should have at least a year's experience.

Mr Campbell had said that he would like to see stiffeners on one side of the girder, and no stiffeners on the outside, but why should not they be omitted on the inside of the girder and put on the outside ? That would in many cases make the erection of the floor very much easier.

Mr West did not appear to agree with the statement in the Paper that concrete was not altogether desirable as an encasement for steelwork. Mr Turton's view, which he had already expressed, was that it was desirable except in special cases to leave the steelwork exposed in places where it could be efficiently painted. In some cases, including possibly such a case as Mr West had shown, it was not possible to paint the steelwork efficiently.

Mr Shirley Smith's word of warning on welding was of value. Some of the points which he mentioned had been encountered, and efforts had been made to overcome them in recent designs. They were all items which were receiving careful consideration and welded design was not being approached blindly. With regard, however, to Mr Shirley Smith's remark that he would like to keep some riveted work, it would be interesting to know which bridges he would decide should be riveted and which should be welded. That would be a fruitful source of argument in a Railway Engineer's office.

Mr Turton agreed with Mr Kerensky that dead and live loads should be properly calculated, making full provision for all forseeable future increases, but so far as bridges were concerned there was always a possibility of unforeseen increases being imposed which would reduce the margin against failure, and there was already a tendency to reduce that margin by designing to higher permissible stresses.

Deterioration by corrosion of a metal bridge could and did take place, no matter how well it was designed and protected, if it was situated in a vulnerable position. The perfect protection against corrosive attack in all atmospheres had not yet been found.

Mr Kerensky's three basic rules for designers were admirable but Mr Turton felt that the two factors which he himself had mentioned should not be dismissed because they could not be accurately assessed. If the designer or engineer responsible for the design considered, from his knowledge of the circumstances, that there was a possibility of one or both

factors operating in the future, he should exercise his judgement in either providing additional material or working to a reduced permissible stress.

Mr Cox, in reply, said that most of the speakers appeared to have concentrated on points raised in Part I of the Paper. That was perhaps some indication of the dilemma with which he had been faced when he came to write Part II ; one could write a great deal about bridges and modern design but " other structures " were more elusive. There were, however, some interesting points which had been raised on his part of the Paper.

Mr Bridge advocated sheeting under timber which was against steelwork, and presumably meant that where it was necessary to put timber against steelwork, one should put down some form of bituminous sheeting. Mr Cox agreed, but suggested that so far as possible timber should not be put against steelwork. Mr Bridge also asked whether there was any shattering where chairs were bolted direct to the concrete. Mr Cox had one small bridge where felt pads had been put down. So far there had been no evidence of shattering, but he disliked the arrangement intensely because it was almost impossible to get a good alignment on the permanent way, however carefully one put the bridge down.

Mr Campbell had underlined the point made in the Paper about the need for designers to have maintenance experience, and had suggested 18 months to 2 years with a bridge inspector or steelwork inspector. Mr Cox heartily agreed with that suggestion but thought the period mentioned somewhat long. He did not, however, agree with Mr Toms, with all due respect to the universities of Great Britain and their products, that a university graduate's training, however sound, fitted him to proceed immediately with design in a way which would avoid all future maintenance problems.

Mr Campbell had joined issue with Mr Cox on two examples quoted of simplicity in design. Mr Campbell was inclined to the nice symmetrical shape of a tee instead of a long-legged angle on a roof truss. Mr Cox had not quoted the latter as necessarily the best form of design, but only to show the advantage of a single unit against two smaller units laid back to back. With regard to Mr Campbell's criticism of the roof work in *Fig. 16*, that had been included merely to illustrate the position of the inspection gangways. Such being the case, Mr Cox did not feel he was called upon to deal with Mr Campbell's criticism of the truss design.

Mr Henry had kindly enlarged on the building shown in *Fig. 8* and had given more information than it had been possible to incorporate in the Paper. Mr Cox was appalled that anybody should have adopted 2½-inch-thick reinforced-concrete panels and steel window frames for a structure in South Wales or in any maritime atmosphere. He thought that those windows should have been timber-framed and built on brick curtain-walls. He had had experience of steel windows in Cornwall bursting the brickwork in which they were set, purely through corrosion.

Mr Shirley Smith had coupled Mr Cox's name with Mr Turton's as advocating welding on all occasions, but Mr Cox did not entirely subscribe to that view. He was not opposed to welding, but in certain circumstances it was difficult to decide whether a bridge should be welded or riveted. He was very doubtful about whether such structures as lock gates should be welded. He understood that some of the bigger firms making lock gates now only quoted for welded work, but from his experience of dock work, including lock gates and caissons, he thought that riveting had certain advantages over welding. He could not claim an exhaustive knowledge of welding on Liberty ships, although he had seen one which had been damaged in a collision and brought into Plymouth during the war, and it was true that the welding had held and the plates had parted. He had formed a very poor opinion of the quality of American steel used in that particular vessel.

The closing date for Correspondence on the foregoing Paper was the 15th May, 1953. No contribution received later than that date will be printed in the Proceedings.—Sec. I.C.E.

ROAD ENGINEERING DIVISION MEETING

3 February, 1953

Brigadier A. C. Hughes, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 40

"Some Aids to Traffic Flow"

by

Richard Albert Belcham Smith, M.C., A.M.I.C.E.

SYNOPSIS

The Author has avoided reference to major schemes the essentiality of which has already been well-explored and concentrates on such aids as might well be given practical execution even during a period of stringent economy. He refers to traffic congestion, the reduction of which might be obtained by better siting of bus stops and by alternative routes especially lit and signed. The value of pedestrian-operated lights, the use of lines of reflectors at the crossings, and contrasting colour at all crossings is examined.

The faults of the existing road junctions are examined and a use of a limited form of traffic subway suggested.

Congested traffic in cities is examined and suggestions made for its relief by a system of outer garages to which is connected a bus service and/or tube.

Outside towns the use of white lines is examined, as also are the design of roundabouts, left and right hand turns, and the gradients of roads at entry to a main road.

Lay-bys and their design for various purposes are discussed and attention drawn to faults in existing ones.

A plea is made for the use of a "built-up area" sign rather than a speed limit sign.

Name-plates and signposting receive criticism and improvements are suggested; the use of the "halt" sign is condemned.

Finally the case for by-passes is pressed especially from the point of view of road safety and national economy.

Introduction

Traffic aids have two main objectives to fulfil, namely, to maintain a smooth and safe flow of motor traffic by day and by night, and to give to users of the road, other than motor vehicles, a fair use of the highway.

There are two sets of circumstances which require separate study: those which occur in highly populated areas and those in less densely populated areas. Each of these is important in its way, but the former is governed by a heavy volume of traffic proceeding at 30 miles per hour and less, and the latter mainly by traffic proceeding at higher speeds.

For the purposes of this examination it has been thought fit to ignore

the present economic situation of Great Britain and explore all the means that could be made available and seek how to use them to the best advantage, leaving it to others to decide the extent to which it is financially possible to put them into practical execution.

Traffic in Congested Areas

The majority of accidents occur in built-up areas and it is here that the greatest number of obstructions to free flow occur. To try to cure these by ideal systems of fly-over crossings and pedestrian subways is delightful to contemplate but quite impossible economically to execute, and less expensive assistance must be sought.

Bus Stops and Lay-bys

The first query that arises is whether or not obstructions can be reduced. In built-up areas, the main roads are occupied by a large number of motor cars, lorries, and buses, of which only the latter have any frequent necessity to stop and start. This suggests that stopping places are an inescapable nuisance and that two-line traffic must inevitably be reduced to one line at frequent intervals. Any bus stop near traffic lights is bound to reduce, at peak hours, the number of vehicles which can get across before the lights change, thus causing impatience and a tendency to " shoot " the lights, which sometimes causes a side road to be blocked and the advantage of the change of lights to be partially lost so far as the side road is concerned. It is suggested that there are many places where up to 4 feet could be cut off the pavement width for the length of several buses without any detriment to pedestrian traffic, and such a 4-foot width would, in many instances, be quite sufficient to allow two-lane traffic to proceed unimpeded (see *Fig. 1*).

The queues for those wishing to board buses could be formed at either end, and the buses might be equipped with a number disc which the driver could swing out so that at busy stopping places the bus is easily identified from the queue.

Alternative Routes

The congestion caused by lorries could be much reduced by better routing. It is easier to take the main route because it is the obvious one. But it may be possible to make the alternative routes more obvious, and it is suggested that this can be done by closer co-operation between the traffic authorities and the commercial transport organizations by the issue of route maps giving alternative routes so that all ambiguity is removed. The system " Follow the amber lights," which has been successfully adopted on the Underground Railways, could be applied by giving road name-plates on these routes a distinguishing coloured background or by many other methods, such as coloured " cats-eye" studs which a fertile imagination might devise.

The start of an alternative route for commercial traffic should be clearly indicated, as is done, for example, at Maidstone. Street name-plates along the route should all have the same coloured background and lettering —the clearly marked route signs of the A.A. and R.A.C. demonstrate the value of this. Distinguishing name plates will fulfil their purpose by day, and by night the stranger would be greatly assisted if alternative routes had a distinguishing coloured form of lighting. For instance, if the general lighting of the main roads is greenish in tint, then a yellowish or amber tint could be used on the alternative routes.

Pedestrian Crossings

To facilitate the movement of pedestrians across the road without interfering with the smooth flow of traffic is, as already mentioned, a financial impossibility ; but a stage has now been reached when the flow of traffic is unduly impeded, especially where some of the so-called zebra crossings have been laid down.

It appears that some of these crossings may have to be equipped with pedestrian-operated lights instead of beacons, combined with an overhead light which would only come into operation when pedestrians are crossing.

There are, of course, a number of busy centres where attractive subways could and should be constructed.

The adoption of white stripes for zebra crossings emphasizes the advantages of light-coloured crossings. On side roads, at their junctions with main roads, the provision of zebra crossings is seldom justified. On the other hand, there is no valid reason why a light-coloured road surface should not be provided which would be to all appearances a continuation of the pavement. This would have the advantage of making the pedestrian more visible at night, even if he were not accorded the same privileges as on a zebra crossing.

It would appear desirable that all important pedestrian crossings should have a parallel line of reflectors, about 10 feet away from the actual crossing, which would be the stop line for vehicles ; this, combined with the suggested pedestrian-operated lights, should make these crossings safe. However, an equally efficient method may already be in the process of development by the Road Research Laboratory.

Research

Mention of the Road Research Laboratory inevitably calls for a word of praise, and if there is room for possible improvement it is only in the time lag between their first examination of a problem and the eventual introduction in practice of the result of their investigations. It would seem that part of this delay is attributable to the narrow limits of research when carried out entirely by any single Department ; it is recommended that any authority should be allowed to undertake an experiment, after notifying the Department, and that as much co-operation as possible should

Fig. 1

Bus Stop which could be Recessed and thus prove More Efficient

Fig. 2

Setts used to indicate Centre of Road

Fig. 3

Inclined Approach to Main Road

Fig. 4

Clear View of Bus Bay, showing Likely Cause of Obstruction

Fig. 5

An Objectionable Type of Kerb

Fig. 6

Bus Obstructing Traffic as it Pulls out from Bay

Fig. 7

A Sign with Insufficient Information

Fig. 8

A Useful Sign

be obtained from all organizations concerned. Road research is a field in which practical knowledge is of equal value to scientific knowledge, and both should be combined to obtain the right answer the quickest way. The remainder of the delay apparently consists of the time required by the Ministry of Transport to examine a recommendation made by the R.R.L., to bring it to the notice of all the authorities and organizations concerned, and to obtain their replies. These remarks apply to all road research and not only to that required under this section.

Road Junctions

Improvement of the numerous cross-roads on main roads cannot be dealt with better than is being slowly done at present, but junctions on secondary roads frequently fail because a proper view of approaching traffic on the cross-road is prevented by obstructive screens such as high walls and hedges.

The pavement sweep should not be too wide because this encourages traffic to take the corner too fast, but the impediment of a high wall or hedge should be removed by cutting off the corner. In many cases this would be allowed by the landlord, provided that the cost were borne entirely by the authority.

Traffic congestion on main roads in built-up areas can nearly always be traced to definite localities, especially in large cities. For instance, Oxford Circus and Marble Arch in London are definite bottlenecks which cause the traffic to pile up for long distances. There is no real cure for this except by construction. If a traffic census were taken it would probably show that by constructing one traffic subway running, say, east and west, the traffic above ground would be so much reduced that the flow would be reasonably smooth even in peak hours. It is often assumed that a vast system of over- and underbridges with sweeping approaches would be necessary at such points to relieve congestion. The Author does not take that view and believes that traffic surveys would prove his point. It should also be realized that the exit of traffic going in one direction need not be at the exact point at which traffic in the opposite direction enters the traffic subway. The argument against such subways is usually that sewers and other services would form impassable obstacles. The Author believes that British engineers could almost always meet this objection in a satisfactory manner.

Waiting Vehicles

It is not intended in this Paper to consider the vexed question of standing vehicles in all its aspects, but since any solution would eventually be a very valuable traffic aid, a review of the general principles might be useful.

Parking space is required for three categories of road users: for shoppers; for office workers; and for commercial delivery traffic.

It is suggested that all new buildings should have garages, but this is

only a partial solution involving a very long-term policy, and could only be applied to large new buildings. The Author advances the following theory for a more immediate practical solution, considering its application to London—since this is the city best known to him. It is not an entirely new theory, but it has never, so far as the Author is aware, been rejected as economically unjustifiable.

So far as the office worker who drives to the office, or the shopper, is concerned, his vehicle will never be continuously at the point where he shops or works; in other words, he must always do a certain amount of walking, as well as riding. Is it justifiable, therefore, to take up road space for his vehicle in the most valuable parts of the town, which will only be occupied for about one-third of a day—from 9 a.m. to 6 p.m.? Only in a theatre centre is the time of occupation longer.

Taking transport from the south-west, west, and north-west as an example, large eight-storey garages could be built near Battersea, Chelsea football ground, Earl's Court, Kensington High Street, Notting Hill Gate, Maida Vale, Marylebone Station, and other such approach points. Each garage would be served during peak hours by a service of buses running right into the garage to pick up and set down passengers. Thus, if it is assumed that every car carried two passengers and every bus carried forty, then the road space of twenty cars becomes available and the road space for one bus only is required; surely this would be a valuable traffic aid and not a great inconvenience to any of the private motor drivers and their passengers, who, by reason of the smoother traffic flow, would reach their destinations in about the same time and in a much happier frame of mind, having avoided the traffic blocks hitherto caused by excessive numbers of private vehicles.

The delivery van is no simple problem; eventually, no doubt, all deliveries in the big modern blocks will be made from service roads probably at the back of the premises, or by roads within the buildings. Initially, however, much more could be done by the firms themselves. If the label on the goods indicated the floor of the building to which the article is to be delivered, it would save many vehicle-hours per week; the really prompt dismissal of the carter with his receipt would save still more time, and the packing of the vehicle should be so arranged that first deliveries come out first.

There is no simple remedy for the problem created by the shoppers. In country towns, waiting on one side only is being adopted more and more, whilst in bigger towns limited waiting usually has to be allowed on both sides of the road. Possibly some benefit would accrue if the parking were to be on the "outward" side in the morning and on the "inward" side in the evening, leaving a clear run for the main flow of traffic. There is the more stringent method of forbidding parking on the main streets, but that cannot be extended until sufficient garage space is provided. The subject of parking is not really within the province of this Paper,

but it is to be hoped that in the new towns which are now being built round London, an example will be set of the way in which parking can be arranged so that the shopper is tempted to the car park by its convenience. Vehicles should not have to re-enter the main shopping street on leaving, and the appropriate exits should be clearly defined.

Street Lighting

With regard to the aids of a more detailed nature which can be usefully applied to roads in towns, the major requirement is the application of good diffused lighting on all main roads, with less powerful but equally diffused lighting on alternative routes; the degree of lighting on other subsidiary roads should be decided by what local authorities deem necessary in the interests of security and of pedestrians using the pavements.

Traffic in Less Densely Populated Areas

The problem of traffic in less densely populated areas is governed by a set of circumstances arising from the speed of motor cars and the weight of heavy lorries, many of which are of considerable width, and restricted to slow speeds.

White Lines

Traffic lanes are usually divided by white lines, setts, or hard fabricated units inset.

Long lengths of white lines are tiring to the driver on long journeys; it is preferable to have such lines laid down intermittently, with a notice every half-mile reminding drivers to keep to the near side. The possibility of providing a central strip of granite setts is worthy of consideration as a means of providing an audible indication of the centre of the carriageway. These have proved very valuable, for they can immediately be felt when a wheel passes over them, even though they may not be quite so easily visible as white lines. A good example of these granite setts is to be found on the Dunfermline–Creiff Road (see *Fig. 2*).

Cats-eye studs are now increasingly employed, both as a centre-line and as a kerb-line warning, and they are particularly valuable at the approaches to, and on, curves.

Road Junctions

The Author is of the opinion that a large proportion of accidents with vehicles on through routes occur at cross-roads and other road junctions. The first essential is good sight-lines in every direction, and it is surprising how few road junctions provide this aid. A left-hand turn into the main road should include a widening of the main road starting at, say, 8 feet at the side road and diminishing to zero in about 100 yards. This would always give the driver on the main road a reasonable chance of avoiding

a collision with a vehicle coming out of the side road carelessly. Traffic seeking to go straight across a main road is usually more wary.

The right-hand turn is a bugbear at all junctions ; a stop line 10 feet wide in the centre of the road might assist, provided that it were coupled with traffic lines at least 200 yards in length to indicate the space clearly as that reserved for traffic turning right, and if sufficient space were given to through traffic on either side to proceed in as many lanes as are provided on the rest of the length. Generally this arrangement would only be required on the main roads where high speeds are prevalent.

On a dual carriageway the position of the vehicle intending to turn right is more easily denoted. If the centre strip is stopped, say, 100 yards short of the turning, then a vehicle can be drawn into the centre area between the two carriageways and cease to be an obstruction while awaiting its turn to cross over, and traffic approaching from behind has sufficient warning of the intention to turn and passes by on the near side with an unimpeded run.

Drivers entering from side roads are handicapped if the rise is steep at that point (see *Fig. 3*), for they dislike stopping on a hill which already calls for a low gear ; similarly, if the fall is steep they may underestimate the safe stopping distance. There are many instances where quite a lot of the immediate steepness could be eliminated, even if it meant an increase in the gradient farther back. These difficulties may not be regarded as such by experienced drivers, but a large percentage of drivers on the road are not swift in their reactions and yet are perfectly entitled to use the road. All traffic aids should indeed aim at making it possible to drive safely on a road even though one's reactions are not quick ; it would be impossible to confine the roads to drivers of, say, 19 to 30 years of age, or to only very experienced drivers.

Roundabouts

At very important junctions, roundabouts have been introduced and in many cases serve their purpose admirably, but it seems obvious that finality of the principles of design has not yet been reached. The term " principle " is used deliberately here, since no standard design could meet the requirements of every set of circumstances. It is unfortunately a fact that existing roundabouts are usually far too cramped. This is mainly because of the lack of area available, but is also often attributable to a lack of appreciation of the new set of conditions imposed on drivers and their reactions to them. The driver is no longer able to go straight across the road, but must now join a stream of traffic which he would normally cut across. But he still has definite intentions, any one of which has now become more complicated, except his turn to the left, and even that is sometimes obstructed because of the narrow width of the roundabout.

The designer should always bear in mind that unless a reasonably free flow is possible a sense of impatience or frustration is aroused, which is

reflected in bad behaviour at the roundabout and an unnecessary increase in speed on leaving it, which in turn may lead to accidents farther on.

The main objection to many existing roundabouts is lack of weaving space. When the traffic is dense in all directions, the only cure is to increase the area available. When, however, the traffic is greater, say, from north to south than from east to west, an elongation of the north-and-south run may help traffic passing east and west to weave with less difficulty. If a vehicle travelling from north to south meets a vehicle practically at right angles which is trying to weave into the inner track, there is either good-humoured obstruction or impatient barging, both of which cause a delay that gradually builds up during the hours of heavy traffic until there is a long queue of almost stationary vehicles. This is economically unsound and is reflected in the costs of both public and private transport.

Although the amount of traffic is very much reduced at roundabouts when there is a spell of fog, the reduced speed enforced undoubtedly causes an equivalent build-up of the traffic queue ; it is therefore essential that the lighting of roundabouts should be such that it gives a definite aid under these conditions. Such lighting may have to be on a separate circuit and only switched on for fog conditions, for the points of illumination will usually have to be at a much lower level and may even be of the infra-red or similarly effective type.

It is not right to discuss the subject of roundabouts without a reference to their use by heavy indivisible loads. The majority of the roundabouts on main roads cannot be easily negotiated—and sometimes cannot be negotiated at all—by vehicles of extreme overall length or when laden with items up to 16 feet wide.

Typical examples of such roundabouts are :—

The Western Avenue–Llandaff Road crossing near Cardiff.
South side of Litchfield on the A.38 Birmingham Road.
The junction of A.614 with A.616, at Ollerton.
The junction of A.616 with A.619, at Barlborough,
The junction of A.38 with A.44, south of Worcester.

All these are on important roads which are used regularly by vehicles carrying indivisible loads. The use of an alternative route may add £100 or more to the cost of transport and give the driver added cares in avoiding pitfalls. A " straight through," guarded with police-controlled barriers, is all that is necessary.

Colonel G. T. Bennett, O.B.E., B.Sc., M.I.C.E., has investigated in a practical manner the types of design for road junctions in general, so it is unnecessary for this subject to be discussed in further detail in this Paper.

Cyclists

In the interests of safety, greater efforts should be made to segregate cycle traffic from motor traffic. Cycle tracks should be made more

attractive, their surface should be maintained in first-class condition, and special attention should be given to the problem of their intersection with side roads. In some industrial areas, it might be more economical to provide a two-way cycle track on one side of the main carriageway only, particularly where development is mainly on one side of the road.

The pedal cyclist is particularly prone to accidents at or near cross-roads. In the Author's opinion, the cyclist on a main road should always be confined to a cycle track for the last 100 yards before a road junction, and then only allowed to wheel the cycle across on a defined track to the far side of the junction. Such tracks could also be used by pedestrians when they cross the main road and could in some places be fitted with " cross now " press-button lights which would also switch on a powerful overhead light or lights, as mentioned previously in connexion with pedestrian crossings.

At road junctions in factory areas, subways with easy gradients and wide enough to take three cycles abreast might be provided, and their use made compulsory unless the bicycle is wheeled across the road at a defined crossing.

Obstruction by Stationary Vehicles

Although many accidents occur in the neighbourhood of road junctions, an unwarranted number also occur in between junctions, and these are often caused by obstructions of a temporary nature. These temporary obstructions are usually vehicles parked at the side of the road by cafés or sports grounds, or at beauty spots (which seem so often to be on roads with many bends), or they may be broken-down vehicles.

The road system of Great Britain is, generally speaking, designed for a free flow of traffic, assuming that no vehicle will ever stop for any purpose whatsoever ; there appear to be few signs of a change from that outlook even at the present day. When a road is widened it is because the flow of traffic is so heavy that the widening cannot be avoided, yet there are long stretches on many roads which would still serve their purpose if obstacles were to be removed. An extra uphill track on steep gradients would enable slow-moving traffic to keep to the left so that faster traffic could overtake.

The next important aid is that given by removing temporarily held-up vehicles on any section of the road.

In April 1943, a Committee was set up by the Minister of Transport to study lay-out in the interest of traffic flow ; their Report was published in 1946. Plate 1 of that Report, showing a waiting-bay for buses at Windsor, was presumably intended to be an ideal example, yet that bay is so designed that a bus must swing out sharply when returning to the main road. (See *Figs 4* and *6*.) Plate 6 in the Report showed a back access lane with loading bays in buildings, yet none of the loading bays appear to be on the skew. The pedestrian barriers shown in the Appendix

to the Report are dirt traps and cannot be cleaned, because no openings for water are provided. Further examination of the Report would reveal other similar short-comings ; this is mentioned not with intent to criticize, but rather to show how much thought must be put into the simplest aid to traffic flow, and to stress that if the matter is to be approached at all, an attempt must be made to get as close as possible to the ideal.

Lay-bys

The design of lay-bys, including their siting, distance apart, and area, has to be based upon the type of traffic on the road concerned.

On a road to, say, Birmingham (an essentially commercial route) the lay-bys must be fairly frequent and capable of housing two lorries comfortably ; it is also suggested that they should be at every half-mile at least, and on both sides of the road. On a road from London to the South Coast, the length of two cars would be sufficient, but in this case the lay-by should be divided from the main road by a grass verge or island, so that the occupants of the car or cars do not stroll unthinkingly on to the road ; again the draw-out should be gradual. It is surprising how few people give any signal when they are starting off in a vehicle. If the lay-by is close to a café, sports ground, or similar attraction, the length should be increased unless facilities are provided off the main road, as they certainly should be. But a lay-by should never be sited close to a road junction, because large vehicles then become an obstruction to the sight-line for drivers coming out of the side road. Recent research and new regulations may reduce the risk of accidents by enforcing the provision of suitable rear lighting, but the only real answer is to get the waiting vehicle off the road, especially at night. It is suggested that, rather than construct an odd lay-by here and there, it would be better to fulfil this aim route by route, concentrating, say, on a road from London to Birmingham and another from Manchester to Preston—two commercial routes carrying very heavy traffic. It would be interesting to learn from the Police and from the transport owners whether such aids were appreciated and had intrinsic value both from the point of view of road safety and free traffic flow. It seems clear that good would result, but actual figures might be more revealing and give encouragement to action on a greater scale.

Kerbs

The use of right-angled kerbs on roads outside built-up areas is generally very undesirable, except where they are necessary for the protection of pedestrians on a footpath. Flush kerbs, or those set at an obtuse angle to the road, are to be preferred, since they enable vehicles to be drawn off the road in the event of an emergency or during a stop for rest, recreation, or repair, and they also encourage drivers to keep closer to the nearside of the road, thus improving traffic flow. Ministry of Transport Memorandum No. 653 states that " where there is a footway a vertical kerb should

be provided; a height of 4 inches is considered suitable. Splayed kerbs or sunken kerbs may be provided for central reservations and along verges where there is no footway." It seems a pity that the word "may" is used—the word "should" would surely be more appropriate. Unfortunately, many authorities are still erecting vertical kerbs on main traffic routes, even where there is no footpath; a recent example of this is to be found on the Great North Road (a trunk road!) south of Biggleswade in Bedfordshire. The vertical kerb in this case is as high as 7 inches in places (see *Fig. 5*).

Speed Limits

The progress of legislation with regard to speed limits is shown in Table 1.

TABLE 1.—SPEED LIMITS

Legislation	Speed limits		Other provisions of Act
	In towns	Elsewhere	
Locomotive Act, 1861 . .	5 m.p.h.	10 m.p.h.	
Locomotive Act, 1865 . .	2 m.p.h.	4 m.p.h.	Required attendant on foot with red flag.
Highways and Locomotives (Amendment) Act, 1878 .	2 m.p.h.	4 m.p.h.	Attendant in front, but red flag no longer required.
Locomotives on Highways Act, 1896 ("The Emancipation Act")	14 m.p.h.	14 m.p.h.	Defined "light locomotives" (under 3 tons).
Motor Car Act, 1903 . .	20 m.p.h.	20 m.p.h.	Defined "motor cars."
Road Traffic Act, 1930 . .	No limit	No limit	
Road Traffic Act, 1934 . .	30 m.p.h.	No limit	

Apart from the general speed limits indicated, which apply mainly to cars, powers existed, and still do exist, to impose special speed limits where considered necessary, or for particular classes of vehicles.

A 30-mile-per-hour speed limit is now enforced in built-up areas, but this and even lower limits are self-imposed on roads carrying heavy traffic, especially in areas where "black spots" are indicated. In other areas the limit is of doubtful value. It is not always kept by lighter vehicles and does not seem to reduce accidents, which, when they do occur, are generally caused by an error of judgement or by dangerous or negligent driving, in which speed is not a primary factor.

This is an example of the type of restriction in which large-scale experiment should precede any legislation. If, say, Bristol, Birmingham, and

Bradford imposed a 30-mile-per-hour limit for a period of 2 years and could show a greater reduction in the number of accidents than, say, Leeds, Liverpool, and Leicester with no speed limit, the former checking drivers for exceeding the limit, and the latter seeking to detect dangerous or negligent driving, some useful information should be obtained. Legislation should not be imposed when based upon guesswork, whatever the pressure brought to bear.

It is arguable, in fact, whether any fixed speed limit is desirable. A safe speed depends upon the traffic conditions prevailing or to be expected, and on the weather, the visibility, and the physical nature of the road. A speed of more than 30 miles per hour might be perfectly safe at some times of the day on a given road, and yet a speed of 25 miles per hour on the same road at a different time might be dangerous. Would it not be better to abolish the arbitrary speed limit of 30 miles per hour, which cannot make allowance for changing conditions, and emphasize, by means of the Highway Code and any other means available, the obligation of a driver to use his judgement in all circumstances rather than to be guided by artificial restrictions? The Police could then concentrate on ensuring a high standard of road behaviour at all times. This would entail a built-up-area sign instead of a limit sign to warn drivers to beware of a greater number of possible causes of accidents.

Street Name-Plates

The value of street name-plates has been admitted—the Ministry of Transport issued Circular No. 340 (Roads) in 1930, a Departmental Committee on Traffic Signs issued a Report in 1944, and on the 28th May, 1952, designs for street name-plates were issued; but so far as the travelling public is concerned much remains to be done. It is stated that the lower edge should be not less than 2 feet 6 inches nor more than 12 feet above ground, and then suggests that in city areas the most suitable height would be about 8 feet.

Only the latter recommendation does not appear to take sufficient account of traffic conditions in cities. Cities generally consist of commercial areas, office areas, and large blocks of flats. In these cases, 8 feet is rarely high enough except when there are no parked vans or lorries; it is suggested that 12 feet, but no higher, would be a much better height.

Signposting

The provision of improved signposting has been as long delayed as the erection of sufficient street name-plates. The Report of the Departmental Committee on Traffic Signs was issued in 1933, there was a further Report in 1944, and the "Traffic Signs (Size, Colour and Type) Regulations" were issued in 1950, consolidating previous regulations.

Route numbering still has its opponents but one suspects that they are diminishing. So far as "A" and "B" roads are concerned, route number-

ing is extremely valuable for long-distance traffic, but cannot take the place of signposting for more local traffic ; nor is it sufficient for the long-distance driver who wants to know not only what road he is on, but where he is and also the point at which he must expect to turn off, which may occur between two villages whose names are scarcely obvious in day time and still less so at night.

The Traffic Signs Regulations, 1950, permit the use of two alternative types of place name-signs (see *Figs 7* and *8*). One of these includes only the name of the town itself, and the other bears at the top the name and distance of the next place of importance ahead, and at the bottom the name and distance of the last important place behind. It is, perhaps, a pity that both types have been authorized, because for reasons of economy the former type of sign is all too frequently used, and it is, of course, much less helpful to the road user. It is understood that one of the motoring organizations has received a great deal of correspondence regarding the gradual disappearance of their circular " village " sign, which was distinctive and included much useful information.

In Eire, on the main roads, the Automobile Association has erected signs at the beginning of each town and village, giving the name of the town which the driver is about to enter ; on leaving the town another sign indicates the name of the next town or village ahead.

Warning Signs

The use of " Slow—Major Road Ahead " and " Halt—Major Road Ahead " signs is not really satisfactory. Careful drivers take heed of the word " Slow " but others are scarcely interested and reckon that they will be able to deal successfully with any situation which may arise, occasionally giving a short blast on the horn. It is suggested that the phrase " Danger—Major Road Ahead " would be more effective psychologically, and in the event of an accident, dangerous driving could almost be proved by the fact that the sign had been ignored. The word " Halt " might then be allowed to disappear, in fact it is unfortunate that such a term was ever allowed. If the places where these signs are placed are examined, it becomes obvious that, in the majority of cases, they could be eliminated by giving proper sighting range to traffic in each direction. Unfortunately, there seems to be a growing tendency for " Halt " signs to be erected at junctions where the line of sight is *not* severely restricted. Surely it is unreasonable to expect a driver to " Halt " if he can see that the road ahead is clear. In such cases a " Slow " sign should suffice—but a " Danger—Major Road Ahead " sign could well replace both types.

By-Passes

Every motor vehicle driver knows how all main roads in Great Britain are cluttered with congestion spots, and what a serious cause of delay and traffic chaos they are. These delays not only cause loss of time but

arouse friction between drivers and a feeling of frustration which is very often reflected, when the congestion is passed, in faster driving than would otherwise be necessary. In the case of road transport the delay is more serious nationally. The driver working to schedule either conforms strictly to the speed limits, in which case he falls behind time, or he increases his speed. If behind schedule, he may either fail to deliver a consignment in time (which is particularly serious if for export, or for a factory short of a particular material), and if he increases speed he not only breaks the law but increases the cost of running and the cost of maintenance. Places such as Stamford, on the Great North Road, in aggregate, add thousands of pounds to the cost of transport which the consumer must pay, and which in the case of foreign competition may mean the loss of contracts.

If it is realized that the delay of only forty lorries for a quarter of an hour *en route* is equivalent to one lorry being off the road for a whole day, some idea of the expensive nature of these delays may be obtained.

The only answer is by-passes, which in many cases could be so designed that they would later be incorporated in a trunk-road system which avoided intermediate towns and villages with great benefit to the latter. Such by-passes would simplify local distribution by the absence of through traffic, and would greatly facilitate access to local shops and offices.

In conclusion, the Author wishes to point out that the Paper was submitted in its present form with the aim of dealing with the situation as it is today and to make suggestions which could, if acceptable, be adopted at once.

Other alternatives were, first, to generalize on the value of major works and demonstrate their value by reference to results in other countries, or secondly, to examine the variety of electrical, mechanical, and other fitments available as aids to traffic. It was concluded that in the first case much excellent information has already been disseminated in the Press to the public, and in the second case the Road Research Laboratory would be in a better position to supply the subject matter.

The Paper is accompanied by eight photographs, from which the half-tone page plates have been prepared.

Discussion

Mr A. J. H. Clayton said that it was interesting to have a Paper written from the point of view of the user rather than that of the road engineer, although he would assure the Author that road engineers did take lorries into account in their designs.

He did not agree that the Author had ignored the present economic situation. The present Treasury policy had of necessity been ignored: no road improvement work was being done at the present time. The

Author had taken into account the present economic situation, in that he had made suggestions not involving a great deal of expenditure and which might do some good, whereas in previous Papers there had been put forward glorious ideals for whose achievement there would not be enough money even if the Treasury provided its share.

Quite properly, the Author had differentiated between built-up areas and the open main roads. For example, the principle of the dual carriageway, in towns, should be applied quite differently in detail from the way it was applied in the country. The essence of a dual carriageway was in having at least two lines of traffic in each direction, not the strip in the middle, and in towns it was sufficient, and perhaps better, if the division was made by means of a sufficient number of refuges.

On p. 417, reference was made to the signposting of alternative routes. He assumed that the Author was not aware of the fact that the London Traffic Committee had had quite a strong programme on those lines in 1926; it had not been very popular with the highway authorities then, and he did not believe that it would be very popular nowadays with the planners because of the routing of traffic through residential streets. That was not necessarily such a bad thing as was supposed. A detailed examination of the accident position in London showed that where traffic was taken through residential streets the accidents per million vehicle-miles were less than when the traffic was taken through shopping streets.

The Author had revived the old fallacy that a road intersection in a town could be improved by driving a tunnel or providing a bridge for one direction (see p. 419). The snag was that there was a lot of turning traffic. A bridge should cater for not less than four lanes—a bridge for two lanes would not handle much traffic. That would require a lot of room, and if the road was already narrow and not handling its traffic adequately, there was not enough room left for the turning traffic.

The Author had mentioned Oxford Circus. That suggestion had been made at frequent intervals in Mr Clayton's experience during the past 25 years. Such a solution there was not out of the question from the engineering point of view, but it was out of the question economically because, in order to carry out the project, it would be necessary to knock down some very valuable buildings. That was an impossibly expensive proposition, to say nothing about the architects' reaction to interference with the four corners of the Circus.

The Author was right to insist on the establishment of principles of roundabout design (see p. 422). Various circulars and books which had been issued had given stock designs for the draughtsmen to apply, but that was very dangerous because draughtsmen tended to apply them without understanding the principles involved, and if the roundabouts were to be built the results would be disastrous.

Mr Clayton thought that the principle of an elongated roundabout had been proposed and subsequently discarded, although there still existed

two elongated roundabouts on the Hampton Court–Esher road and two on Eastern Avenue. One of the principles of roundabout design was that the traffic could only weave if it was brought to the same speed. The elongated roundabout design facilitated the passage of the major-road traffic but did not give the minor-road traffic a chance.

The Author had suggested that the 30-mile-per-hour speed limit was enforced in towns, particularly in London, but Mr Clayton believed that it was not enforced. It would constitute an interesting piece of research to ascertain whether there was any difference in the practice of drivers and in the accident rates, in places where the 30-mile-per-hour speed limit was enforced compared with places where it was not. He was of the opinion that in some towns the speed limit was enforced to a greater extent, or at least obeyed to a greater extent, than it was in London. Perhaps the Road Research Laboratory would care to put that subject on its long list ?

Having been to the United States of America, the Author had suggested that the parking problem there was not quite the same as that in Great Britain. Mr Clayton agreed with that, although Great Britain was approaching the American state of affairs ; for example, something would have to be done about double parking which was now occurring frequently. The idea of having car parks outside the built-up area or the central area had been tried by the London Transport Executive—one had been built, further out than the Author suggested, at Morden—but he did not know whether that car park was fully used now. The people who came up to London in cars were those who did not want to walk to the station or the bus stop at either end, and he doubted whether such car parks would obtain much custom.

The principle that, to be any good, car parks should be within a few hundred yards of the destination of the owner-traveller, was one which was more likely to bring results, although, as in America, it was virtually impracticable economically.

Colonel Sydney Green said that one of his preoccupations was the study of traffic conditions in inner London : he had read with interest the section of the Paper dealing with traffic in congested areas.

Although he did not entirely exclude the possibility of constructing bus lay-bys, and pedestrian and vehicular subways in inner London, he felt that such projects required to be given a good deal of thought. He could assure the Author that a detailed traffic census *had* been taken at Oxford Circus and Marble Arch periodically over the years, and had been studied very carefully. He felt that his colleagues would not allow the occasional sewer, however large it was, to become an impassable barrier to the carrying out of desirable major improvements.

The subsoil of the London streets was full of surprises. To the uninitiated, one of the most attractive sites for a traffic subway was the intersection of Parliament Street and Parliament Square—until it was

realized that the District Railway ran diagonally across the intersection just beneath the surface.

The Author's suggestion for a single-line traffic subway running east to west at Oxford Circus would, presumably, have the sole object of eliminating the effect of the right-hand turn from the west into Regent Street, but that turn could be eliminated in another way. The traffic could be made to turn left before it got to Oxford Circus and the right-hand turn could thus be displaced to the north. Alternatively, the traffic could be made to go straight through Oxford Circus, and then turn left and left again, and so across the Circus from north to south. The Author's proposal for a single-line vehicle subway for east-west traffic, even if it were possible, would have to be balanced against the prohibition of the right-hand turn in the manner described.

Assuming that all the difficulties as to mains and sewers could be overcome—although formidable—the siting of a single-line traffic subway would take up between 18 and 20 feet (allowing for kerbs and the construction of retaining walls) out of a carriageway width of 48 feet. That would leave carriageways on either side of the vehicle subway of 14 to 15 feet, which would be barely wide enough to take one standing line and one moving line of traffic, and it had to be remembered that through the bottleneck created by that restricted carriageway had to pass the whole of the east-bound traffic which contained the right-hand-turn traffic.

The Author had suggested a similar traffic subway passing from east to west at Marble Arch. Only about a third of the west-bound traffic in Oxford Street was proceeding directly west, the remainder proceeding into Park Lane, Hyde Park, and Edgware Road. That third of the west-bound Oxford Street traffic represented only about 6 per cent of the total traffic circulating at Marble Arch, and therefore about 94 per cent of the traffic would still be left circulating on the surface. Apart altogether from the problem of having a bottleneck in Oxford Street, it would be preferable, Colonel Green suggested, to apply the available resources to the enlargement of the roundabout system on the surface, as was proposed in the Boulevard Scheme included in the County of London plan, than to use them for the provision of such a subway.

The suggestion that traffic-subway entrances and exits need not be at coincident points, but in line ahead, suffered from the same disadvantages in that bottlenecks were created. Moreover, in the case of Oxford Circus, to have two subway entrances and exits on each side of the intersection in line ahead meant that the restricted area would extend about 350 yards on either side of the Circus. Two-thirds of the carriageway over that distance would have to be stripped of its mains and other services—a formidable task. Another important point was that the buses which constituted about 23 per cent of the traffic using Oxford Circus would, if constrained to use such a subway, have to drop their passengers about 350 yards away from that busy intersection.

On p. 417, the Author had referred to the obstructive effect of bus stops near traffic lights. *Any* stationary vehicle near a controlled intersection had that effect, and Colonel Green would draw attention to the Report entitled " London Traffic Congestion " produced by the London and Home Counties Advisory Committee in 1951. On p. 32 of that Report, reference was made to an experiment which had been conducted by the Ministry of Transport, and followed up by the Road Research Laboratory, on the effect of standing vehicles at various distances from intersections. In the conditions of that experiment it had been found that a standing vehicle about 100 feet from an intersection reduced capacity by only 7 per cent, whereas a standing vehicle 15 feet away from the intersection reduced capacity by 29 per cent.

The elimination of standing vehicles of all kinds near intersections, either by regulation or widening for them (by arcading where practicable) seemed to be a sounder proposition than the construction of bus lay-bys, particularly between intersections.

He could not understand why the Author had chosen 4 feet as the distance for recessing pavements, particularly when one considered that some of London's buses were now 8 feet wide, and the principal traffic streets seldom provided for more than two bare lanes of traffic in each direction.

Mr H. N. Ginns said that no other country offered so wide a choice of routes, but alternatives were generally inferior to the popular one, which was the most used simply because it was the best. Colour indication of routes failed because of the wide choice of routes and because colour combinations were limited and advance signing was cumbersome. Number combinations were not limited and advance signing of them was simple ; hence route numbering was the only practical method.

Forcing traffic on to less suitable routes would mean spending money on them, thus dissipating resources. The Maidstone alternative route quoted illustrated that ; it was a series of country lanes passing over a bridge 10 feet wide, and through a cutting 14 feet wide, and disturbed four peaceful villages. Money spent on it would be wasted because plans already existed, and land was already bought, for a proper by-pass to the town.

Alternative routes were doubtful expedients and, if the economic factors were to be ignored, ought not engineers to forget expedients and to urge the provision, by improvements and new construction, of roads fit for modern traffic ?

It should become a legal requirement that every new building should absorb the traffic it attracted. Floor space would have to be allotted, and car ramps and lifts installed, and owners would have to accept the extra cost just as they already accepted extra costs resulting from height, structural, fire, and sanitary by-laws. Local Authorities had some powers under Section 17 of the Ribbon Development Act, 1935, regarding

buildings, and a few buildings were dealt with under them. Would it not be sound policy to seek wider powers on those lines ?

Parking on main streets could only be stopped if sufficient free parking was available. Police could not and would not force motorists off main roads into car parks where charges were made. As the law stood, it was essential that car parks should be free.

He suggested greater use of suitable advance signs for car parks where conditions allowed, the signs commencing on the outskirts of small towns and being repeated at intermediate points in large towns.

Could the Author quote evidence that white lines made drivers tired, and give details of the percentages and types of drivers ? That discomfort was surely outweighed by advantages to many, especially at night and in fog. " Keep to near side " boards warned slow-moving traffic on multi-lane roads, but lines helped all traffic and were essential for the driver who never knew quite where he was on the road.

Statistics would confirm the Author's opinion as to the dangers of junctions. For example, 48 per cent of the accidents in Kent in 1951 occurred at junctions. Mr Ginns agreed that where right-turning vehicles left a main road there should be space for them to wait and also for moving vehicles to pass. Acceleration and deceleration splays, recommended in various publications, helped, and at T-junctions the main road could be widened opposite the side road to allow following vehicles to pass to the left of right-turning vehicles. In his experience, even a 30-foot carriageway was too narrow for safe turning on high-speed roads.

Work had been, and was being, done in providing lay-bys on particular routes in many areas. It was his view that there should always be lay-bys at telephone kiosks, including A.A. and R.A.C. boxes, especially on commercial routes, and that they should be signposted.

The word " may " in Ministry of Transport Memorandum No. 653 (. . . " Splayed and sunken kerbs may be provided. . . .") was put in deliberately because such kerbing was not desirable everywhere ; it depended upon subsoil and drainage. In Kent, flush kerbs were satisfactory ; on Bedfordshire clay they were not. As to high kerbs, foundations were often left to consolidate under traffic, and the final surface when applied left a normal kerb face of 4 inches. The raised kerb in *Fig 5* was necessary for drainage and also because a footway was to be added.

There was evidence to show that the introduction of speed limits in Great Britain in 1934 had resulted in reduced accidents, and any wholesale revision of the present system, or its abandonment, would seem politically impossible. Opportunity for studying the effect of removing restrictions occurred in West Germany where general speed limits had recently been abandoned. There was also a field of study in Northern Ireland and some Continental countries where there were no general limits. Was a comparison between accidents in those countries and in Great Britain practicable, and had such an investigation ever been made ?

In advocating by-passes, engineers should not overstress time saved by individual vehicles—it might prove very small. In one case over a 10-mile length, including a bottleneck town, the time gained was no more than 4 minutes for lorries and 6 minutes for cars. But, expressed in terms of total traffic, the saving was important; that by-pass would save about 1,000 commercial-vehicle-hours per week and 6,000 car-hours per week.

Finally, the effect of by-passes on accidents should not be overlooked. In one typical market town Mr Ginns had investigated, a by-pass might save 56 accidents per annum. It had been said that accidents could be reduced by the three E's—education, enforcement, and engineering. Of those, the last had never been adequately applied, and road engineers had not been able to make the full contribution to road efficiency and safety of which they were capable.

Colonel G. T. Bennett observed that the Author had said that he was making due allowance for the economic conditions in Great Britain today. The proposals which he had put forward did, however, include adequate parking facilities in large cities and by-passes, and both those would, no doubt, cost a lot of money. For the most part, the rest of the Paper dealt with detail.

Colonel Bennett thanked the Author for the complimentary remarks concerning himself and his work on road junctions, but he wished that the Author had nevertheless included in the Paper all that he thought about junctions. What Colonel Bennett had done had been most tentative and a beginning only, and such conclusions as he might have reached were by no means yet agreed by others. There was still much to be done.

The Author had suggested that it might be advisable that the traffic should be slowed down at road junctions by using small radii of curvature. Colonel Bennett said that on the whole there was a tendency today to depart from that view; in fact, sometimes it seemed that free flow should be given at a junction, although in other circumstances it should not, and the whole matter was complicated.

It had been suggested in the Paper, which was meant to be read by highway engineers, that if landowners were approached politely, they would allow the highway engineers to pull down their garden walls and build them back 20 feet or so. That had not been his experience, unless the wall was already falling down and likely to become a liability to the landowner. Usually the highway authorities had to build the wall back at their own expense and pay for the land, and when they took a tentative agreement of that sort to the district valuer, he probably thought it was not a very good bargain. Highway engineers knew how difficult were the problems involved in trying to set back corners by acquisition of land, and if all corners were not yet set back for vision, it was not because they did not know it should be done.

He agreed with the Author that kerbs on rural roads should as a rule

be splayed, being cut back at an angle of 45 degrees or so, so that traffic in any trouble could get off the road, and that became more necessary the narrower the carriageway was. He also agreed with the Author as to the great value of lay-bys. He had provided many in the course of his career on various important roads, and it was a source of gratification to see how much they were used by the traffic. It was quite clear that the heavy traffic in particular appreciated lay-bys, especially if they were provided at uniform intervals, so that if one was found to be full it would be known when another might be expected.

Colonel Bennett said that there was no doubt about the need for extra traffic lanes on long gradients. In the case of narrow roads, the widening of several straight lengths in order to give an additional traffic lane, even though it might not be a continuous widening, would give the slow and fast traffic a chance to sort itself out, and congestion would thus be relieved. That expedient ought to be borne in mind in cases where a project to widen the whole length of a road could not be afforded.

However, it was not much good thinking of any expedients at the present time. Even the Author's smaller suggestions—quite apart from his two larger ones—could not be carried out because nothing at all could be carried out at the moment It was a most unfortunate position, but it was to be hoped that it would soon be remedied.

Dr G. Charlesworth said that the Author had stated in the Paper that he aimed at dealing with the situation on the roads as it existed today and at putting forward suggestions which could be adopted at once. That approach to the traffic problem was a useful one for, with the present restriction on capital development, the role of minor improvements in the efficiency of existing roads became much more important; there was no doubt that much could be done by those methods and that some ideas put forward by the Author were worthy of trial. However, it was fairly certain that many of those minor improvements could be better planned if there were available more information about the travelling habits of the public; for example, where people wanted to travel, why they travelled, and how they travelled. Information of that kind was needed when alternative routes were being considered, when deciding on schemes for parking accommodation, and when estimating the economic returns of particular improvements.

The Author had referred to research and the need to reduce the time lag between the start of an investigation and the introduction in practice of the results of the research. The Road Research Laboratory was constantly trying to increase the rate at which research could be completed, but, as in so many things, a major difficulty was shortage of staff. There were only about sixty people engaged on research in the Traffic and Safety Division, and that Division dealt with investigations on skidding, lighting, vehicle and road-user characteristics, as well as on traffic and road lay-out.

Referring to the limits of research, Dr Charlesworth believed that the Road Research Laboratory was unique, considering the very wide range of road research it covered in one organization. The Author had recommended that any authority should be allowed to undertake an experiment. Some, of course, did so, and Colonel Bennett's work on junction design in Oxfordshire was a case in point. At the Laboratory it was desired to encourage engineers to make their own investigations, but the Laboratory would, of course, like to know the results obtained. By collecting together the results of experiments in different localities, it might be possible to arrive at more generally useful conclusions.

It was, of course, important to link research and practice. The Laboratory was trying to evolve practical solutions to road problems, and it worked in close touch with the Ministry of Transport and other authorities. Also, through the Road Research Board and its committees, the Laboratory obtained advice on its work from people of wide experience in matters connected with roads and traffic.

In discussing pedestrian crossings, the Author had referred to the impedance to flow at zebra crossings. It was to be expected that delay to drivers would increase if more of them gave right-of-way to pedestrians on zebra crossings, but it should be noted that accidents to pedestrians had decreased markedly since the zebra crossings had been introduced. Furthermore, since the number of crossings had also been reduced, it did not follow that delay in the aggregate had increased; indeed, it could have decreased. The Road Research Laboratory was looking into matters of delay and was investigating the conditions under which zebra crossings, light-controlled crossings, and subways were justified.

Mr J. T. Duff observed that the Author had advocated the use of pedestrian-operated lights. That idea had been tried, but the number of sites where the vehicular and pedestrian flow was so continuously heavy as to ensure success was very few. The difficulty at the majority of sites, and even at those sites which were heavily trafficked during off-peak hours, was that pedestrians were not subject to the same laws as were drivers, in that they were not compelled to obey the signals. After the button had been pressed, they were apt to cross at the first gap in the traffic, and when the " cross now " signal appeared, all the vehicles were held up for no apparent reason. That was very irritating to drivers, and the result was that greater delays were imposed than in the case of the zebra crossings.

To make pedestrians more visible at night, a light-coloured background on the far side of the junction was required, and not a narrow strip under their feet. There were two objections to the proposed line 10 feet from the crossing. First, a line of reflectors across the carriageway gave the appearance of an obstruction, which could be most confusing to drivers; secondly, a stop line in advance of the crossing would impose a legal obligation which drivers would find it very difficult to obey. A vehicle

could not be stopped instantaneously, and it was not easy to anticipate the movements of pedestrians.

He agreed with the Author's remarks on p. 421 about the importance of good street lighting. The requirements suggested would be found to be met by the recommendations contained in the final Report of the Departmental Committee on Street Lighting, which had been issued in 1937.

The proposal, on p. 422, to cut back the central reservation on a dual carriageway for 100 yards on either side of a junction would give rise to the wide open spaces which were so confusing to drivers.

He agreed that many existing roundabouts were far too small, but disagreed with the suggestion that islands should be elongated in the direction of the heavier flow. As Mr Clayton had pointed out, that had been tried before the war, but so far from making weaving easier for the minor stream, it made it much more difficult, and it increased the dangers by making the speeds of the two streams different. Equality of speed of all the vehicles in the stream was the essence of successful roundabout working.

On p. 423, the Author had referred to the use of infra-red radiations. They were invisible to the human eye, and the Author did not explain how they were to be employed to light a roundabout. Did he propose that all vehicles should be equipped with infra-red image-convertors? If so, would it not be better to equip them also with infra-red headlamps so that other obstructions besides roundabouts could be seen in fog? But why bother with infra-red radiations when the technique of radar was so far advanced?

He could not agree with the Author's suggestion, on p. 428, that a "Danger—Major Road Ahead" sign should be used. That would be contrary to the recommendations of the 1944 Traffic Signs Committee. It would be illogical if there was no vehicle near the junction on the main road, for then there would not be any danger. The present system gave the driver the credit of being a rational creature who, in general, merely needed to be told that a major road was ahead. The word "slow" was perhaps superfluous. However, at a smaller number of junctions the driver required to be told that the visibility was so limited that he could not emerge safely without first coming to a stop in a position where it was just possible for him to see the major-road traffic, and hence the need for the "Halt" sign.

Mr E. T. T. Hussell said he presumed that the object of the Paper was to provoke discussion on the existing aids to traffic flow.

In spite of what he considered to be inconsistencies, he believed that the Paper would achieve its object. Anyone who read it would surely at some stage ask himself: "Are we too conservative in our approach to this subject?" Few would be prepared to give an emphatic "no" or an unqualified "yes" to that question. Present aids to traffic flow had been developed over a long period of years, and many of them had

been in use for so long that there was perhaps a danger of their being accepted because everyone was used to them. Therefore, perhaps it was a good thing that someone should come along and ask such a question as : " Are speed limits really necessary ? " After all, it was for engineers to find an answer to the question, and the exercise might be good for them.

The Author had said that legislation which was based upon guesswork should not be imposed. Although the speed limit had been fixed in 1934, it still had to be confirmed each year in Parliament, and to that extent it was still on trial. On the last occasion the Minister of Transport had said that he proposed to introduce legislation to make the speed limit permanent, and that was after 20 years of trial. He felt that the Author had scored a point there about the time taken in trial and research ; but he lost it again on the charge of lack of thought. Incidentally, it was significant that the number of pedestrians killed after the introduction of the speed limit in 1934 fell almost at once from 3,500 to 3,000 a year, and in 1951 the figure was about 2,400. Therefore, that had been quite a good piece of guesswork.

In 1952 the Minister had decided to introduce what were now commonly known as " flashing beacons " at pedestrian crossings. If any innovation cried out for universal rather than partial adoption, that was surely it ; but the Government had made it optional, and it was understood that the Westminster City Council, in the heart of London, had refused to install them. If some pedestrian crossings were to be lit and others were not, there was bound to be trouble, and, indeed, the last state might well be worse than the first.

Referring to place name-signs, Mr Hussell said that, broadly speaking, the sign with the mileages was used for towns which could be found on small-scale maps. The smaller sign was rather more for the interest of the traveller than for the purpose of assisting him on his way. Graveley was a hamlet on the Great North Road between Baldock and Stevenage, and in his opinion it would be quite wrong to attempt to include it or similar places in the mileage system. The question of cheapness was not really the dominating factor.

The Author had referred to maps only once, and then only very casually, although he had said a lot about routeing of traffic. Mr Hussell wished it were more generally recognized that direction signs were only half the story—the other half must be obtained from maps. That was a field for more intensive propaganda.

Mr S. B. Thomas said that a principal object of the planning and designing of roads was to provide for free and safe flow of traffic, and he used the word " traffic " in its widest sense, including pedestrians as well as all forms of wheeled vehicles. It was clear that the measure of success which attended the endeavours to promote free and safe movement of the traffic, would be indicative of the efficiency of the road system and show an economic return for the money invested in it.

The ideas expressed in the Paper seemed to him to accord with such a thesis and were not in dispute. They found expression in the terms of reference of the Departmental Committee set up by the Minister of Transport during the war years whose report on the " Design and Lay-out of Roads in Built-up Areas " would be well known.

Whether traffic movement was examined from the mathematical aspect of Newton's laws of motion or from the hard-won experience of a school playground with its free-for-all disorder, the conclusion would be that turbulence begot bumps and free flow was short-lived. The ideal was, patently, orderly stream-lined movement devoid of conflict, and that should always be kept well in mind. That ideal could hardly be achieved if obstructions were condoned, and the Author had drawn attention to the various ways in which obstruction could occur.

Restriction of movement as a means of securing safe and free flow was a commonplace ; the use of kerbing to confine vehicles to the carriageway, but, strangely, not pedestrians to their footpath, was an obvious example. There were other forms of restriction imposed to aid traffic movement which were more irksome—he had in mind " No Waiting " orders. Their sign was a yellow-banded post, and it was as irritating at times as that insect with corresponding markings which buzzed around the jam dish on summer picnics !

The Author had made some suggestions for removing the waiting or parked vehicle ; it would have been useful if he had pursued the subject further. The waiting vehicle was evidence of inadequate use of the land adjoining the carriageway or fronting the road, and an attempt should be made to remedy that through control of development. It could equally well be regarded as misuse of a road.

The function of the road was clearly defined in paragraph 144 of the Interim Report of the Committee on Road Safety, published in 1944. The attempt to use the road to provide access to land fronting on to it as well as for the needs of through traffic was a prime cause of the turbulence of movement already mentioned, and that meant obstruction and accidents. It was necessary to determine which of those two functions a road was to serve. Even if the road had, unfortunately, to try to do both jobs with indifferent success, the inherent difficulties could be eased and traffic movement facilitated by intelligent control of development.

That was essentially a long-term approach to the problem, but it had been a long time building up and it would take a long time to resolve. At the least, the New Towns had had a chance, by their new start, to achieve a degree of success barely possible in congested, ill-planned cities.

A moment's reflection on the inevitable concomitants of development would make the point clear. The daily activity of inhabitants and workers required the attendance of tradesmen, municipal services, travellers, and business callers each with a waiting vehicle ; there were stray dogs and bus stops, vehicles making turning movements across the

road to get into and out of premises, and the inevitable presence of children, apart from adult pedestrians. All contributed their quota of interference with traffic movement and introduced hazards where none should be. Prevention of that would indeed aid traffic movement.

Mr G. P. Manning said that the term "speed limit" had been referred to. He remembered the time when several manufacturers of cars guaranteed that their vehicles would exceed 60 miles per hour; at the same time, the law absolutely forbade any vehicle to proceed at more than 20 miles per hour. Had there been any progress from those ridiculous days? He was told that the Government now confirmed the speed limit of 30 miles per hour; it had been stated in the Press—and never denied so far as he was aware—that less than 3 per cent of all private cars and less than 1 per cent of all fully-powered motor-cycles conformed to the 30-mile-per-hour limit. What, then, was the present British speed limit? It appeared to be a mere fantastic delusion which existed solely in one's imagination. The Author had suggested various experiments; it would be a good experiment actually to have a speed limit, for there never yet had been one in Britain! How could the observance of such a limit be ensured? First, the present 30-mile-per-hour signs were not sufficiently distinctive, not sufficiently numerous, and many of them were badly sited.

Next, the present type of police car was quite useless in controlling traffic. What was required was an inconspicuous car driven by a policeman not in uniform. The car required only two articles of equipment—a reliable speedometer and a repeating camera, that was, a camera which had a strip of film and took ten exposures at intervals of exactly one second. When a car overtook the police, the driver would put his finger on a button to operate the camera. The first exposure of the camera would show the speedometer of the police car registering 30 miles per hour and the overtaking car just coming into view in the windscreen. The next exposure would show the overtaking car coming further into view, the speed of the police car still being shown as 30 miles per hour; the exposures would continue to be taken until, finally, the overtaking car would be shown right in front of the police car, the speed of the police car still being shown at the bottom. That not merely established that the overtaking car was exceeding the limit; by measuring the height of the overtaking car in the photograph, one could come very close to ascertaining its actual pace.

With regard to the question of "Halt—Major Road Ahead" signs, Mr Manning thought that it seemed very clear to engineers that if a sign said "Major Road Ahead" anyone should understand what he ought to do. But in fact those signs had to be appreciated by every road user. He recalled a case during the war when a member of one of the women's Services was driving a staff car. She came to a major road where the sign said "Halt—Major Road Ahead." The girl put her brakes on

immediately and came to a dead stop, dropped into low gear, and then proceeded across the major road; a lorry was within 20 feet of her as she came out on to the major road, and its driver was unable to miss her. Yet she had actually halted and obeyed the sign. It had to be realized that one had to cope with drivers of that type. The sign "Danger—Major Road Ahead" was certainly not redundant—not for some types of driver, at any rate.

The Chairman did not agree with Mr Manning's remarks about the police; he thought that police cars should be coloured red and black so that one would always know where they were.

He believed that the speed limit achieved a useful object. It certainly did not keep people down to 30 miles per hour, but it certainly made them drive more slowly. An average speed of 40 instead of 60 was a help in built-up areas.

He was surprised that no one had mentioned, as a factor in checking traffic flow, the insularity of the British which discouraged the adoption of the Continental style of road signs. Living in a sea-board county, he had had experience of what happened to motorists from the Continent who came to Britain; they were frequently found stopping on the road and asking what the signs meant. Of all the countries in Europe, Britain had a separate set of road signs. That was no fault of the Ministry of Transport; it had come about because one of the *ad hoc* committees which sat from time to time had advised the Ministry that Great Britain should have different road signs from those on the Continent. The result was that motorists from the Continent did not understand the British signs, and British motorists visiting the Continent did not understand the signs in France or Germany.

*** **Mr T. S. Sinclair** referred to the Author's comments (on pp. 424 and 425) on the Report of the Committee on the Design and Lay-out of Roads in Built-up Areas.

That Committee, upon which Mr Sinclair had served, had been a strong one so far as highway engineering and traffic was concerned, and its terms of reference had gone further than to make recommendations on the free flow of road traffic—they had specified safety, economy, and the requirements of town planning.

With reference to Plate 1 of the Report, there were two schools of thought on a design for waiting-bays. One was to design them as shown in *Figs 4* and *6*, whilst the other, and the minority view, was to introduce a tapering acceleration lane for the bays. Those points had been debated at some length by the Committee and a preference had eventually been indicated for the first type, since the opinion was held that vehicles stationary in any recesses or emerging from any entry should wait until there was a gap in the traffic on the adjacent lane before entering the main

*** This contribution was submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

carriageway. In fact the design suggested in the Report for that bus bay, or temporary lay-by, was precisely the same as provided on the lay-bys along main roads throughout the country, which were so greatly appreciated by drivers and, so far as Mr Sinclair could see, by the Author.

Plate No. 6 of the Report showed a back access lane where loading of vehicles could take place without interference with the shops along the shopping frontage. Mr Sinclair suggested that it was for the designer of the building and for the occupier to decide whether loading of vehicles should take place at right angles to, or on the skew to the line of the lane, and he could not see that a remark such as the Author's would make any constructive contribution to design of back access roads.

With reference to the pedestrian barriers shown in the Appendix to the Report, those designs had been considered at great length by some very eminent city engineers who had served on the committee, and they had been designed to give maximum protection to pedestrians—one of the major traffic flows which had not been mentioned by the Author—and to show that pedestrian barriers need not be ugly tubular erections. It might be admitted that the solid barriers would probably trap papers, bus tickets, and the like, but in a built-up area where scavenging would be adequate Mr Sinclair thought that the safety of pedestrians would take pride of place. He would mention also that those designs were not shown in sufficient detail for the Author to criticize them on the grounds of lack of openings for surface water.

Just as the Author had made certain presumptions, Mr Sinclair thought he was entitled to assume, despite the Author's remarks that further examination of the Report would reveal similar shortcomings, that the points made were his major criticisms of the Report, and to conclude from that that the Committee were to be congratulated on producing a document of more than a hundred pages and nearly five hundred paragraphs on which only those minor points of criticism arose.

The Author, in reply, said that he had written the Paper deliberately to provoke discussion. It had not seemed to him that he should write about matters in which all were agreed, and he had felt that it would be very much better to offer some thoughts on which there would be differences of opinion.

The greatest amount of opposition to anything that he had said appeared, unexpectedly to him, to come from the Ministry of Transport. He supposed the Ministry had been so dragooned by the economical situation into opposing anything which would involve expenditure that it simply had to take that attitude. He had not in any way criticized what the Ministry had done yesterday or today, but had simply asked it to do something tomorrow.

In recommending that 4 feet should be cut off the pavement width for bus stops, he had not meant that that should be applied everywhere ; it would apply to country roads in particular. The Paper applied not only

to London, but to the country as well. Four feet or so would probably just make all the difference on country roads, which were usually about 22 feet wide. It might be that 6 feet in some cases, or 3 feet in others, would be necessary to give two free tracks. What the distance should be was a matter for the judgement of the highway engineer.

He had recently visited the Road Research Laboratory and had seen a good deal of its work, although he had not had time to see all of it. It was obvious that the Laboratory suffered under a great handicap which he had not appreciated. Questions were shot so fast into the Laboratory that it could not possibly tackle them all immediately. Either the Laboratory should have more staff or there should be some form of priority for them. It would be preferable to give the Laboratory more staff.

It was obvious that the flashing-light proposal in connexion with " zebra " crossings was going to open criticism in some quarters. That was a case—he might have to eat his own words—in which action had probably been taken too quickly. The result of an experiment in one street would not necessarily apply everywhere, and so there ought to be some judgement as to whether such a method should be used. He questioned whether it was really necessary that the lighting provided should be a flash, but that was purely a personal opinion. When a number of flashing lights had been installed and it had been proved that they worked, the Ministry could be satisfied that everybody else would want them and that the public would demand them. If they were proved unnecessary or another method seemed preferable, he hoped that the Ministry would then continue their experiments.

With regard to his suggestion for elongated roundabouts, he was not particular about that. If authorities had the land—he thought the complaint was that they had not—then larger roundabouts would accomplish his object in providing more weaving space. All that was wanted was a roundabout when weaving could take place without obstruction.

The difficulty of introducing minor road traffic into an elongated roundabout had been mentioned. It was true that that difficulty had been experienced, but the Author suggested that if the design at the entrance of the minor road had been such as to give a wider sweep, then that difficulty would have disappeared.

The " Danger—Major Road Ahead " sign had been criticized. Mr Duff was happy that there should be a 30-mile-per-hour speed limit, and yet he was willing that motorists should do more than 30 miles per hour at night. That was illogical, and if there were to be road laws, they should be logical. "Danger—Major Road Ahead" warned a driver that there might be danger in front but not inevitably. A sign saying " 30 m.p.h." merely indicated that the driver must not travel at more than 30 miles per hour. A warning sign indicating a built-up area or an accident-prone area equally warned a driver that he should be careful. Such a notice would not necessarily be the only one; there could be others farther on. At any rate, the driver

would know what one was getting at, namely, that he should drive extra carefully. That was worth very much more than a law which all agreed was broken.

There was no mention in the Paper of underground garages and other off-street parking facilities simply because the Author had not wanted to expand the Paper into one dealing with parking, which was a huge subject.

With regard to the subways, also criticized by Colonel Green, the Author readily conceded that an enlargement of the Marble Arch roundabout would meet the situation, but there seemed to be no ground-level solution to that at Oxford Circus and he could not accept entirely the arguments put forward against a subway, it would still appear to him to be possible to design the subway without pulling down buildings and without taking up so much roadway space as was suggested. The majority of passengers did not wish to board or alight at Oxford Circus but only nearby, the position of the bus stop did not, therefore, need to be very close to the Circus. There were many places in the Provinces where a subway such as was suggested could be more easily incorporated and prove of the utmost benefit.

With regard to the waiting or parked vehicle referred to by Mr Thomas, it did not seem likely that the matter could be solved unless there were parking authorities with powers to build and finance from public sources with which to pay for the construction. The suggestion that the cost should be borne entirely by owners of property did not meet with British ideas of justice, and the cost of office space, etc., would rocket.

With regard to Mr Sinclair's comments, the Author agreed that the Report on the Design and Lay-out of Roads in Built-up Areas was excellent, but it did not follow that the improvements were not possible as the result of experience.

The lay-bys provided on trunk roads were a boon, yet it was felt that the tapering acceleration lane would lead to greater safety; some drivers would not wait for a gap or failed to realize that a vehicle was rapidly approaching from just behind just when they were starting off. Nor was a starting off signal always given. That might be bad driving, but why not eliminate the possibility of its having any effect?

The provision of a straight loading bay was the most economical form of construction and was adopted where there was room to manoeuvre, but when the lane was used by several occupants, that space could be made available only by obstructing other traffic.

The Royal Army Ordnance Corps had staggered their loading bays during the war and had gained much benefit thereby, since obstruction of moving vehicles had been reduced to the minimum. The matter of obstructing other people's vehicles might not be sufficiently studied when a loading bay was being constructed along a back street open to the public.

With regard to white lines, the principle of a demarcation of that order was very readily acceptable, but the Author suggested that a line of setts

was equally as effective and required no maintenance ; in addition, the eyes of a driver on long distance work did not become fatigued to the same extent as when being continually caught by a white line which appeared to be continually flashing by.

The closing date for Correspondence on the foregoing Paper was the 15th May, 1953. No contributions received later than that date will be printed in the Proceedings.—Sec. I.C.E.

AIRPORT ENGINEERING DIVISION MEETING

3 March, 1953

Sir Hubert Walker, Member, Chairman of the Division, in the Chair

A Ballot was held for the election of five members of the Divisional Board. A list of the members of the complete Board will appear in the Proceedings, Part III, August, and Part II, October, 1953.

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Airport Paper No. 23

"Methods of Soil Stabilization and their Application to the Construction of Airfield Pavements" *

by

Donald John Maclean, B.Sc.,‡ and Peter John Mackenzie Robinson, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

Consideration is given to the use of stabilized soil for constructing the bases of airfields of the permanent or of the temporary military type. As shown in the Appendix, a fairly large number of airfields have already been constructed with stabilized soil, the techniques employed being usually extensions of road practice.

Owing to the large increase in the wheel loads and tire pressures of modern aircraft this procedure can no longer be adopted, and, as with all other forms of base material, it is essential to design the structure of the airfield pavement in relation to the stresses imposed by the aircraft wheels and the strength of the stabilized soil and of the subgrade soil. It is shown that stabilized soil should always be regarded as forming part of a flexible pavement, and methods of determining the minimum stability it should possess in different circumstances are suggested.

The relevant properties of the materials produced by mechanical, cement, bituminous, and chemical methods of stabilization are discussed in relation fo the requirements they must satisfy in airfield construction, and a description is given of full-scale trials in which the performance of soil-cement for use on military airfields has been investigated.

Introduction

In the period since the end of the war (1945) there has been a considerable increase, in the United Kingdom and in many other countries, in the proportion of new roads being constructed by some form of soil stabilization process. A more conservative policy has usually been adopted towards the application of soil stabilization to the construction of airfields, although —as the Tables in the Appendix show—its use has not been entirely neglected, particularly in the United States of America. It appears that

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the techniques used to design and construct airfields by soil stabilization processes have been largely extensions of road practice, and little attention appears to have been given to the problems involved in meeting the particular requirements arising in the case of airfields.

Since 1947, the Road Research Laboratory of the Department of Scientific and Industrial Research and the Military Engineering Experimental Establishment of the Ministry of Supply have collaborated on a research whose primary object was to develop methods of soil stabilization that could be applied to a wide range of soils, and in all types of climate, for the construction of temporary roads and airfields required for military purposes. The research is still in progress, but sufficient results have now been obtained to warrant certain conclusions being drawn concerning the extent to which stabilization processes can be applied in airfield construction. Although the requirements for temporary airfields differ in certain respects from those for permanent airfields, there is enough common ground to enable equally valid conclusions to be drawn in respect of the latter type of airfield.

In this Paper consideration is first given to the basic principles involved in the design of airfields employing stabilized soils, and this is followed by a discussion of the properties of different types of stabilized soils, with particular reference to their ability to meet the various design requirements. The Paper concludes with a description of certain aspects of the joint research being undertaken by the Military Engineering Experimental Establishment and the Road Research Laboratory, and details are given of some of the more important conclusions that have so far emerged.

The Case for Soil Stabilization

The case for the use of stabilized soil as a construction material for airfields is based either on the fact that the soil on the site comprises of the order of 90 per cent of the material required to construct the base, or that use can be made of a local low-grade material that can be more easily obtained than orthodox materials of high quality. For temporary military airfields the first consideration is paramount, since the alternative of winning and transporting granular material in the quantities required would be often impossible and in any case would be a lengthy task of great complexity. In the case of permanent airfields this consideration does not apply, and it is on the grounds of cheapness of construction that either of the two alternatives should be used in preference to the more orthodox methods of construction.

To illustrate these matters, the case will first be considered of a military airfield 2,400 yards long and 50 yards wide, where a base 6 inches thick is required. The volume of material required for this base is 20,000 cubic yards; for a granular base all this material would have to be imported, whereas with a stabilized soil base only of the order of 2,000 cubic yards of

material would have to be imported, the remaining material being the soil on the site. As a second example it is of interest to consider the order of saving in the cost of constructing a permanent airfield with runways, perimeter track, and hardstandings having a total area of 500,000 square yards and requiring a base 18 inches thick. In post-war road construction in the United Kingdom, stabilized soil has on an average been cheaper by 4*d.* per square yard per inch thickness than the next cheapest form of flexible base material such as hardcore. Using this figure in the example quoted, a saving of £150,000 would result. Considering the use of unreinforced concrete in the example, whilst a thinner construction could be employed, this is offset by the fact that, on the average, stabilized soil has been about one-third the cost of the same thickness of concrete.

A further advantage of soil stabilization processes is that they can now be carried out entirely by machine without the necessity, for example, of such preparatory work as form setting and provision for joints in concrete works; this leads to high rates of construction and low manpower requirements. The mix-in-place method, in which the soil is treated where it is situated on the ground, is particularly effective in this respect, since the handling of the soil is reduced to the minimum. With modern equipment, rates of construction exceeding 5,000 square yards per day can easily be achieved by this method of construction.

It is thus considered that there is a strong case for the use of soil stabilization for constructing airfields, if the resulting material can be demonstrated to be sufficiently strong and durable for the purpose. So far as temporary airfields for military purposes are concerned, the responsible authorities have done much to sponsor research into this subject, but it appears that the potentialities of using soil stabilization processes for constructing permanent airfields have not been fully realized. It is hoped that the information contained in this Paper will lead to careful consideration being given to this matter.

Function of Stabilized Soil in the Airfield Structure

Temporary Military Airfields

During World War II, temporary military airfields were usually constructed simply by laying a prefabricated surfacing directly on a prepared soil formation. Although this form of construction was generally adequate for supporting the aircraft then in use, the wheel loads and in particular the tire pressures of aircraft have increased to such an extent that an airfield with the necessary stability can now be obtained conveniently only by the provision of a base between the prefabricated surfacing and the soil.[1] For the reasons stated in the previous section, it is considered that this base can most conveniently be constructed by stabilizing the in-situ soil.

Two types of prefabricated surfacing have been employed on forward

[1] The references are given on pp. 477 and 485.

airfields—metal surfacings and bituminized hessian surfacings. The function of the metal surfacing is to reduce the shearing stresses set up in the soil by the wheels of aircraft, and thus reduce the resulting deformation of the soil; unfortunately, all surfacings of this type that have been developed so far are pervious to rainfall and also permit high losses of moisture by evaporation. The resulting loss of stability of the soil owing to weather largely offsets the gain in stability resulting from the rigidity of the metal surfacing. Although there has been a considerable improvement in the design of metal surfacings since the war, this fundamental weakness has so far not been overcome, and metal surfacings placed directly on soil do not thus provide an airfield structure of high stability in areas where the climate gives rise to extensive moisture variations in the soil.

The function of the prefabricated bituminized hessian surfacing is to provide an impervious cover to the soil formation to prevent increases in moisture content from rainfall that would result in a loss in stability of the soil, the aim being to lay the surfacing when the soil is in a dry and stable condition. Investigations of the moisture properties of soils have shown that a loss of stability of dry soil will occur after the surfacing is laid, owing to the rise of moisture from the ground-water table, the rate being dependent upon the permeability of the soil. It is therefore not possible by this method to maintain the highest stability the soil is capable of developing. With the increase in the wheel loads and tire pressures of aircraft, the stability likely to be obtained with many soils is inadequate, and there is therefore a need in such cases for the provision of a base of more stable material.

The present position with regard to temporary military airfields is therefore that greater stability is usually required than can be obtained with either type of prefabricated surfacing. To some extent the increase in stability could be obtained by the development of a surfacing which combines the rigidity of the metal surfacing and the imperviousness of the bituminized hessian surfacing. Such a development has yet to be made, but in any case there is a need for a strengthened formation under the surfacing. The general form of temporary military airfield is thus visualized as a stabilized soil base with a prefabricated surfacing.

Permanent Airfields

The structure of permanent airfields is usually the same as that of a conventional type of road—namely, a surfacing with a base and possibly a sub-base—and the stabilized soil can be employed as the material for constructing either the base or the sub-base or both, the surfacing being generally of the bituminous type. Since the strength required of a satisfactory sub-base material is considerably less than that required of a base material, there is thus scope for the use of stabilized soil with a wide range of stabilities depending upon the function the material is required to fulfil in the airfield structure.

In considering the possibility of using stabilized soil in the construction of permanent airfields, the responsible engineer may be influenced by the fact that the bituminous surfacing required on the stabilized soil is liable to damage by spillage of fuel and by the high temperatures of the exhaust gases from aircraft with jet engines. This is undoubtedly a serious problem, but it is understood that some promising new surfacing materials have been developed recently, based on synthetic resins or coal tar pitch.

Design Considerations

In considering the problem of designing an airfield structure which incorporates a layer of stabilized soil with a surfacing of bituminous materials, it is first necessary to establish whether the structure will behave as a " rigid " or as a " flexible " base, that is, whether the stabilized soil acts in a similar way to concrete or to a material such as a hardcore or crushed stone.

The essential difference between the two types of base is that the rigid base has sufficient flexural strength to enable it to bridge local areas of the subgrade which are not affording full support to the base, whereas the flexible base has a small flexural strength with the result that the base remains in full contact with the subgrade at all times.

Considering the behaviour of the rigid base in more detail, it acts as a perfectly elastic material when subjected to stress conditions for which it has been designed. Thus, after the passage of a loaded wheel the rigid base recovers completely the deformation it possessed under load. The deformation of the subgrade, on the other hand, is generally not entirely recoverable, and some permanent deformation will result from compaction, consolidation, or plastic flow of the soil. After a number of repetitions of load, the rigid base is generally not fully supported by the subgrade and continues to behave as a continuous slab because of its high flexural strength (exceeding 500 lb. per square inch). If the flexural stresses exceed the strength of the material, cracking will occur, and with a material of high flexural strength the cracks are usually spaced at distances of at least several feet and are often sufficiently wide to reduce considerably the degree of interlocking between the two sides of the crack.

With a flexible base the material has a very small flexural strength similar to that of a soil, with the result that after the application of a loaded wheel the base assumes a permanent deformation equal to that of the subgrade.

Considering now the properties of different types of stabilized soil in common use, only cement-stabilized soil possesses a significant flexural strength, the values in the case of mechanically stabilized and bitumen-stabilized soils being negligible. The latter two types of material thus by definition behave as flexible materials, but the function of soil-cement needs closer examination.

The flexural strength of soil-cement corresponding to an unconfined

compressive strength at an age of 7 days of about 250 lb. per square inch, which is a common strength required of the material when used in road bases, is of the order of 50 lb. per square inch. One method of considering whether soil-cement with this strength acts as a flexible or as a rigid material is to estimate the thickness of base that would be required to prevent a flexural failure. Using the Westergaard method of analysing stresses in a concrete slab, a thickness of soil-cement of 24 inches would be needed on a subgrade with a modulus of reaction of 100 lb. per square inch for a wheel load of 9,000 lb., compared with a thickness of 6 inches for concrete with a flexural strength of 500 lb. per square inch. Since much smaller thicknesses of soil-cement—for example, 6 inches—have proved successful in practice it follows that flexural failure must usually occur so that the soil-cement acts as a flexible base material. The formation of cracks in soil-cement after the passage of traffic has been observed in a number of cases (*Fig. 1*, p. 466). Where the unconfined compressive strength was about 250 lb. per square inch at 7 days, the cracks were very fine and occurred at spacings which divided the material into pieces of the size of hardcore. Since the cracks were fine, the degree of interlocking between the two sides of the cracks remained sufficiently high for there to be no large loss of stability, and the material owed its stability to the close interlocking between individual lumps in the same way as with hardcore and crushed stone bases. Measurements showed that the California Bearing Ratio (C.B.R.) of a soil-cement base, in which hair cracks had formed under load, exceeded 300 per cent; as shown later in this Paper, this value exceeds that obtained with a highly stable granular base.

The observed occurrence in soil-cement of closely spaced fine cracks is considered to be a result primarily of the flexural strength of the material. A rigorous analysis of this phenomenon has not been attempted, but a much simplified explanation is afforded by considering a road base to be composed of a number of discrete beams of equal size. If the beams are simply supported and subjected to a uniformly distributed load it can be shown that the length which will just fail to support the load is linearly related to the square root of the flexural strength. This will be the case with soil-cement where the length of the contact area of the tire is comparable with the observed spacing of cracks. In the case of concrete the beams can be regarded as being subjected to a central point load since the spacing of cracks is much greater than the length of the contact area of the tire. The length of beam which will just fail in these circumstances is linearly related to the flexural strength. Thus, for concrete with a flexural strength of 500 lb. per square inch and soil-cement with a flexural strength of 50 lb. per square inch, the ratio of the spacing of cracks in the two materials would be about 70 : 1. In spite of the many factors which are ignored in this simplified explanation, this ratio is in close agreement with observations and seems to provide a reasonably conclusive explanation of the phenomenon.

It is concluded therefore that soil-cement and all other forms of stabilized soil are flexible materials, and that when they are used to construct bases covered with a bituminous surfacing their thickness and stability should be determined by a method of flexible-pavement design such as the California Bearing Ratio method. It is not proposed in this Paper to deal with any possibly controversial issues concerning the relative merits of different methods of design, and it is probably sufficient to point out that the available experimental evidence suggests that the type of orthodox base material has a negligible effect on the thickness of structure required. It follows that, in the present stage of knowledge, stabilized soil bases should be constructed to the same thickness as that required with any other type of base material.

Criterion for Stability of Stabilized Soil

An important aspect in the design of the runways and perimeter tracks of airfields incorporating a stabilized soil base is the selection of suitable criteria for the minimum stability which the material must have in order to fulfil satisfactorily its function in the structure. The material used in a base must have a much higher stability than the subgrade soil, since it has to withstand the repeated application of much higher stresses. As described later in the Paper, the procedure adopted in tests made to determine the strength of the base material must be suitably adjusted to allow for the effects of climate and ground-water conditions.

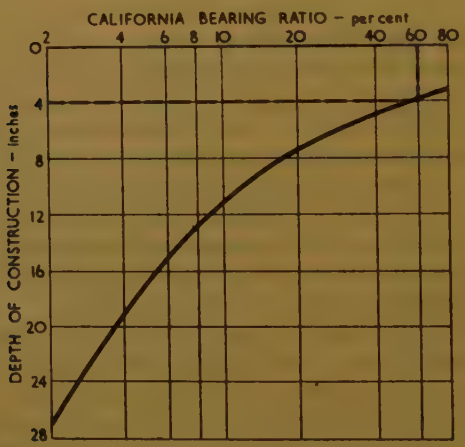
The two principal tests used in Great Britain for measuring the strength of stabilized soil are the test associated with the California Bearing Ratio method of pavement design and the test for measuring the unconfined compressive strength. Both these tests have been standardized by the British Standards Institution and are described in B.S. 1924 : 1953.

(1) *California Bearing Ratio test.*—In the test associated with the C.B.R. method, the load required to cause a circular plunger approximately 2 inches in diameter (cross-sectional area 3 square inches) to penetrate usually 0·1 inch into the stabilized soil at a constant rate of 0·05 inch per minute is expressed as a percentage of the load required to cause the same penetration in a standard material with a C.B.R. of 100 per cent. The standard material which was employed by the originators of the test was a well-compacted sample of crushed stone which is known to be a satisfactory base material, when covered with a bituminous surfacing, for roads carrying heavy traffic. Reference to the C.B.R. design curve for main roads (*Fig. 2*) shows, however, that a base is required to have a C.B.R. of only 60 per cent to give a satisfactory performance when it is covered with a bituminous surfacing 4 inches thick.

There is insufficient field evidence to enable the C.B.R. requirements to be precisely specified for base materials used in permanent airfield construction. The problem can, however, be approached qualitatively by considering the relationships between maximum shear stress and depth for

a range of wheel loads and contact pressures. Approximate distributions of maximum shear stress have therefore been calculated by means of the Boussinesq analysis, which assumes the elastic behaviour of the materials, and these are reproduced in *Fig. 3*. This shows that the maximum shear stress occurs at the surface and is proportional to the contact pressure and independent of the load, but that, for any given contact pressure, the maximum shear stress on any given horizontal plane is determined by the load. In general, therefore, the tire pressure will determine the type and thickness of material used in the surface layers, and the load will determine the total thickness of structure required on any given subgrade. It is

Fig. 2



Design Curve for Main Roads, California Bearing Ratio Method, showing C.B.R. Requirement For Bases under 4-inch Bituminous Surfacing

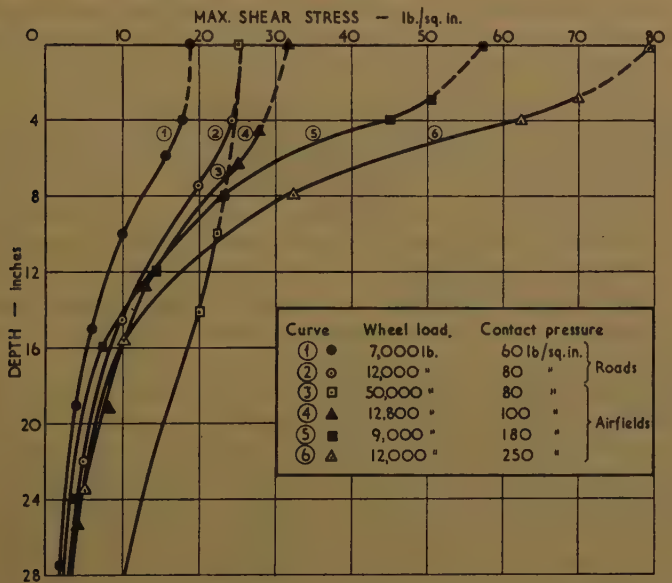
worth noting that in the case of temporary military airfields, where thicknesses of construction are necessarily reduced to the minimum, the whole construction corresponds to the upper layers of a permanent airfield and tire pressure assumes a greater importance.

As a result of accelerated traffic tests at Barksdale in the United States, Hansen [2] concluded that a base having a C.B.R. of 80 per cent gave satisfactory performance when subjected to wheel loads of 50,000 lb. with a contact pressure of 80 lb. per square inch, when covered with a bituminous surfacing 3 inches thick. Reference to curves 2 and 3 in *Fig. 3* shows that this conclusion is broadly in agreement with the road case mentioned above, since the maximum shear stress at a depth of 3 inches (curve 3) is similar to the stress at 4 inches (curve 2), each being about 25 lb. per square inch.

Curve 6 of *Fig. 3*, which gives the distribution of maximum shear stress resulting from a load of 12,000 lb. and a contact pressure of 250 lb. per

square inch, suggests that, even if the thickness of bituminous surfacing is increased to 6 or 8 inches, the C.B.R. of the base must be considerably greater than 80 per cent, since the maximum shear stresses at these depths are 1·8 to 1·3 times as large as the stress of 25 lb. per square inch which can be accepted by a material having a C.B.R. of 80 per cent. Whilst the C.B.R. requirements for base materials used in pavements built to carry wheels exerting these high contact pressures cannot be estimated accurately

Fig. 3



VARIATION OF MAXIMUM SHEAR STRESS WITH DEPTH IN UNIFORMLY ELASTIC MATERIAL SUBJECTED TO PNEUMATIC-TIRED WHEELS WITH DIFFERENT LOADS AND TIRE PRESSURES

from present knowledge, it is considered that a suitable criterion might be a C.B.R. of 150 per cent under a surfacing 6 inches thick.

In the selection of a suitable criterion for the stability of a stabilized sub-base for permanent airfields, it is necessary to choose a C.B.R. value which is sufficiently higher than that of the subgrade of C.B.R. 3 to 10 per cent to justify its use, but sufficiently lower than that of the base to enable a reasonable thickness (say, 6 inches) of the latter to be employed. It is considered that a C.B.R. of 20 per cent satisfactorily meets these requirements.

In the case of temporary military airfields it is difficult to suggest a criterion for the stability of stabilized base which will be covered with a prefabricated surfacing since there is not enough information on the extent

of surface deformation which can be permitted before aircraft find it difficult to take off and land. It is considered that some deformation must be permitted, since this factor more than all others will enable a substantial reduction to be made in the amount of construction needed for temporary military airfields. In the absence of the necessary information on this point it is suggested that a C.B.R. of 50 per cent should be employed as a criterion for the present.

(2) *Unconfined compression test.*—In the unconfined compression test use is made of cylindrical specimens with a 2:1 length/diameter ratio. The diameters of specimens are 2 inches, 4 inches or 6 inches, according to the size of particles contained in the specimen. This test has been used mainly to investigate the suitability of soils for stabilization with cement, for which it is usual to measure the strength of specimens cured at a constant moisture content and temperature for 7 days. For secondary roads, a minimum laboratory strength under these conditions of 250 lb. per square inch has been found to be a suitable criterion for ensuring satisfactory performance of the material in the road.[3] Bearing in mind that in road works the strength of soil-cement at a given age has rarely been higher than 50 per cent of the corresponding laboratory strength, it is considered that for secondary roads a field strength at 7 days of 125 lb. per square inch has proved adequate for a soil-cement base covered only with a surface-dressing.

On this basis and again considering the shear stresses given in *Fig. 3*, it is possible to suggest criteria for the compressive strength of soil-cement when used to construct bases for airfields. For permanent airfields it is considered that the minimum field strength at 7 days of the soil-cement base should be 250 lb. per square inch, when used under a bituminous surfacing 4 inches thick to carry aircraft with tire pressures not exceeding 100 lb. per square inch, and a minimum of 450 lb. per square inch under a 6-inch-thick surfacing for aircraft with much higher tire pressures. For sub-bases for permanent airfields a 7-day field strength of 125 lb. per square inch would be quite adequate.

In the case of military airfields covered with a prefabricated surfacing, a minimum strength of 200 lb. per square inch is suggested.

(3) *Weathering effects.*—Whether the C.B.R. or the unconfined compression test is used, the effect of climatic factors can be examined by suitably modifying the test procedure. The principal factor is probably the softening effect which can occur in bases under a pervious surfacing. This is usually simulated by immersing the specimens in water for a period before measuring their strength. Under an impervious surfacing the base material in many climates tends to an average moisture content which depends upon the ground-water conditions.[1] Since this moisture content governs the stability of the material, an estimate has to be made of the probable moisture content and the test carried out on the material in this condition. In the United States, tests have been standardized, in con-

nexion with soil-cement, in which the stabilized soil is either alternately wetted and dried or alternately frozen and thawed.[4] In these tests the subsequent strength is not measured, but observations are made of the damage caused by the two types of treatment. Experience in the United Kingdom suggests that these durability tests are unnecessarily severe in most climates when the stabilized soil is covered with an adequate thickness of bituminous surfacing, but they might be more relevant in appropriate climates to temporary military airfields where the stabilized soil is partially exposed directly to the effects of weather.

Methods of Soil Stabilization

Having dealt with the function of stabilized soil in the construction of airfields and with the various design factors that are involved, it is now proposed to consider the four main groups of stabilization processes. They are :—

(1) Mechanical stabilization.
(2) Stabilization with cement.
(3) Stabilization with bitumen.
(4) Stabilization with chemicals.

Consideration is given to their principal characteristics and to the extent to which they meet the design requirements for airfields. Brief reference is made in the Appendix to the applications of soil stabilization to airfield construction that have been noted from various sources. In this connexion it appears that although little stabilized soil has been used to construct airfields in the United Kingdom (concrete having been the principal form of construction), soil stabilization processes have been fairly extensively used abroad.

Mechanical Stabilization and Granular Bases

Mechanical stabilization is a term that has been used to describe, in a general way, methods of construction in which soils or soil-like materials are selected or artificially produced to give a base material that is strong enough in a compacted state to give a stable base without the use of special admixtures. However, it is considered that it is unnecessarily restrictive to consider mechanical stabilization of soils as a process differing from the construction of granular bases as a whole, since the principles involved are essentially the same as those involved in the construction of flexible bases with any granular material.

Thus, three factors influence the resistance to shear of a compacted mass of particles. They are :—

(*a*) That fraction of the internal surface area of the mass over which the particles are in contact with each other.

(*b*) The internal friction between adjacent particles, which is a function of the amplitude and frequency of their surface irregularities.

(*c*) The cohesion between the individual particles, which is usually caused by the presence of water films, but which may also arise from natural cementing agents.

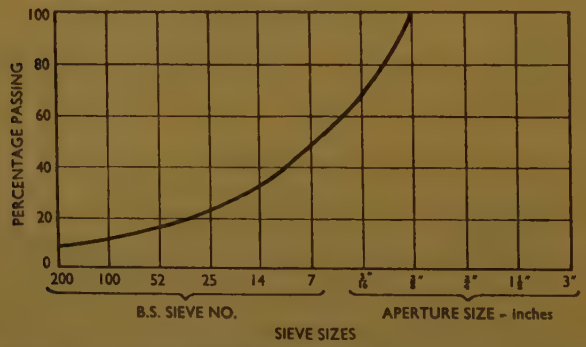
The internal friction of a compacted mass of particles is a function of the area over which the particles are in contact, and will consequently increase as the area per unit volume increases. In both the mechanical stabilization of soils and the construction of bases with crushed hard igneous rock, the area of contact of the particles is increased by using a distribution of particle sizes such that the maximum volume of the voids between the particles of larger size are filled with smaller particles. This has given rise to numerous specifications for the particle-size distribution of such materials in which the grading curve runs from coarse to fine sizes and is concave upwards, resembling that given by Fuller's equation [5] :

$$P = 100\sqrt{\frac{d}{D}}$$

where P denotes the percentage smaller than a given size d
and D ,, the maximum size of particle used in the mixture.

Fig. 4 shows the grading corresponding to Fuller's equation for $D = \frac{3}{8}$ inch.

Fig. 4

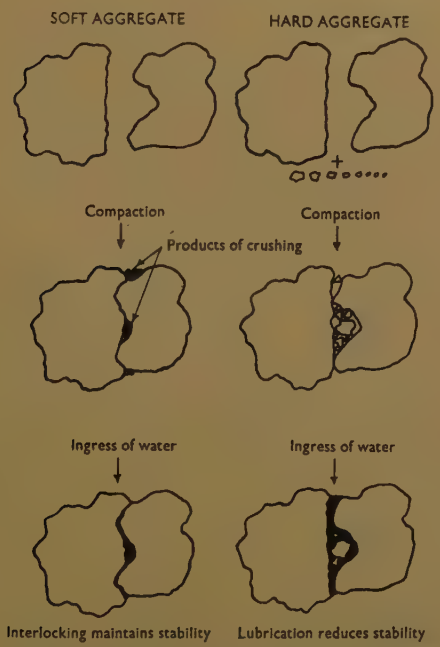


Size Distribution Curve derived from Fuller's Equation, for Maximum Size of Particles of $\frac{3}{8}$ inch

A high degree of internal friction is thus a characteristic of all materials used in granular bases, and the extent to which it influences the stability of the material appears to depend largely upon the surface texture or irregularity of the individual particles after compaction. It is possible to draw a distinction of degree between the behaviour of hard aggregates,

such as crushed igneous rocks, and that of the softer stones, such as limestones, chalk, coral, and laterite. With igneous rocks the internal friction developed is due solely to the surface texture of the individual particles produced in the crushing mill (*Figs 5*), whereas in the case of the softer materials, where compaction crushes individual particles together to produce close mechanical interlocking, the surface irregularities have a large amplitude and a lower frequency, and the irregularities on the surface

Figs 5



DIAGRAMMATIC ILLUSTRATION OF PRINCIPLES GOVERNING THE BEHAVIOUR OF MECHANICALLY STABLE MATERIALS

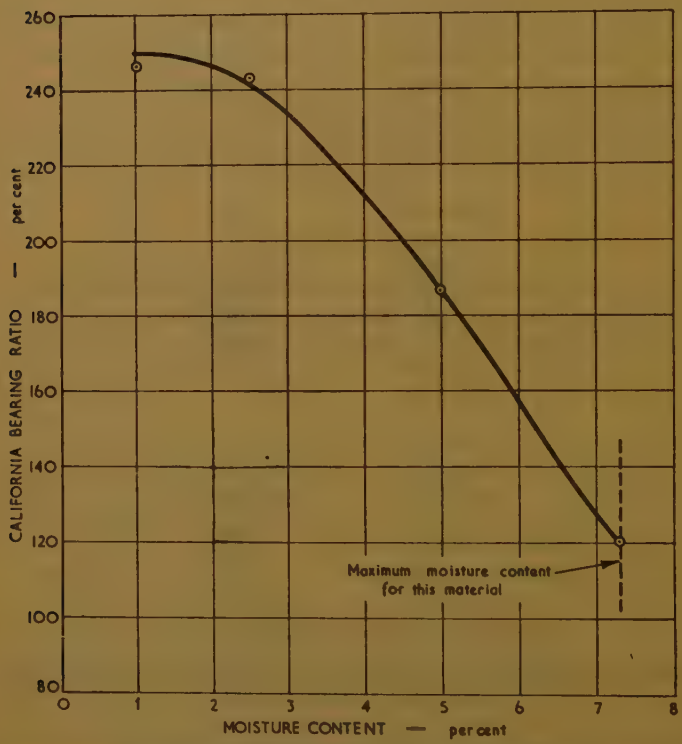
of one particle synchronize with those produced on the adjacent surface of a contiguous particle to give a strong material at all moisture contents, as illustrated by the C.B.R. values for a crushed limestone shown in *Fig. 6*.

A practical result of this difference is that a base constructed with an aggregate that is mechanically weak has a greater stability under load than a similar base constructed with a mechanically strong igneous rock, even where precautions have been taken with the latter to obtain a satisfactory particle-size distribution. This will be particularly the case when the moisture content of the material is high, since the smaller surface irregularities of the igneous rock particles are more easily lubricated than the

larger interlocking areas of the softer stone. There is a lower limit, however, to the strength permissible in individual particles, which is governed by their resistance to complete disintegration under the combined effects of loading stresses and weathering.

In the case of mechanically stable soils, the necessary strength is derived not only from the internal friction but also from the cohesion arising from the films of moisture surrounding the finer particles, particularly the clay

Fig. 6



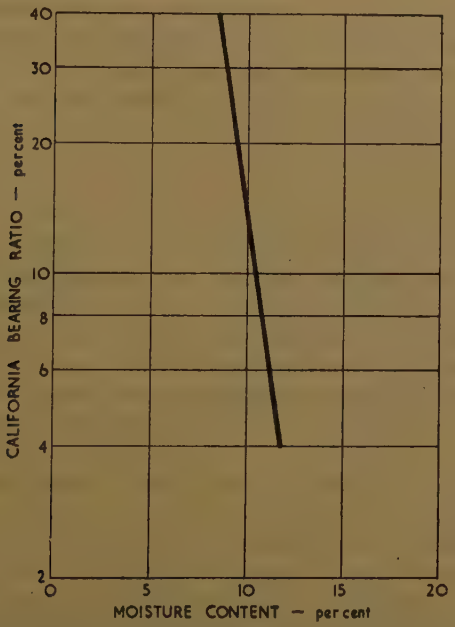
VARIATION OF CALIFORNIA BEARING RATIO WITH MOISTURE CONTENT FOR WELL-COMPACTED CRUSHED LIMESTONE

particles. Care must be taken, however, to avoid excessively high moisture contents in such cases, since when the films of moisture become thick there is not only a decrease in cohesion but also of the internal friction, as a result of the lubrication of the areas of contact of adjacent large particles. This effect becomes particularly serious when the internal friction developed by the larger particles is low, that is, where alluvial gravels with rounded particles are used. To illustrate this, *Fig. 7* shows the effect of moisture

content on the C.B.R. of a sandy gravel. In practice, this means that close control must be exercised over the proportions of clay present in the mixtures, and this is usually achieved by the specification of the maximum plasticity index that can be tolerated in the soil mortar. An analysis of failed and successful gravel bases undertaken at a number of airfield runways in the southern states of the U.S.A. has shown that, to prevent failure of the base material in shear, the plasticity index of the soil mortar should not exceed 5 per cent.[6]

Little is known about the cohesion arising from the presence of natural

Fig. 7



Effect of Moisture Content on the California Bearing Ratio of Sandy Gravel

cementing agents that are believed to occur in certain types of calcareous and ferruginous low-grade aggregates. It is believed that bases laid with such aggregates and left unsurfaced tend to develop a structure due possibly to the solution and re-deposition of soluble constituents such as calcium carbonate and iron oxide during alternate periods of wetting and drying. However, if such cementing effects are in fact associated with movements of moisture or changes in moisture content, it seems probable that they would be unlikely to occur in bases covered with an impermeable surfacing.

Practical application of mechanical stabilization.—There are two methods by which the principles of mechanical stabilization are applied in

practice. In the first method, a suitable soil is mixed with the soil on the site in the proportions necessary to obtain a well-graded material which, when compacted at the optimum moisture content for the compaction plant, gives a stable layer. It is emphasized that proper compaction at an appropriate moisture content is an essential feature of this type of stabilization, since if the moisture content is unduly high or if the compaction is inadequate the material will have a low stability. If a base of satisfactory stability is laid by this method, it is essential in wet climates to cover it with an impermeable surfacing, and the ultimate stability of the base material is then determined by the moisture content it will attain as a result of the upward movement of water into it from the water table. The composition of the material should be such as to give a maximum stability in these conditions. This form of treatment is only likely to justify itself when the soil on the site is a poorly graded gravel which requires only a small proportion of the finer soil fractions to bring it to the required grading. With fine-grained soils it is simpler to place imported mechanically stable material directly on the soil.

The second method of applying in practice the principles of mechanical stabilization is to import suitable materials to the site to form a stable layer placed directly on the subgrade. Gravel deposits can be used but considerable control must be exercised over both the composition and the moisture content if satisfactory results are to be obtained. During construction in wet climates the gravel-sand-clays may be preferable to the clean sandy gravels, in that they are fairly impermeable when compacted and therefore maintain their stability reasonably well, whereas the sandy gravels are permeable and are liable to become unstable even after compaction.

A highly stable base can be obtained with crushed limestone or other similar materials which crush under the action of a heavy roller. It has been found that the best practical method of using this type of material is to lay the coarse stone ($2\frac{1}{4}$ inches to $\frac{1}{2}$ inch) first and to bind this with finer material to form a dense mass after rolling. The final grading and moisture content do not seriously affect the ultimate stability. When a hard igneous rock is used, more care must be exercised over the grading of the material, and the moisture content plays a more important part in affecting the ultimate stability obtained.

It has been mentioned that good compaction is an essential feature of mechanical stabilization. The heaviest smooth-wheeled rollers are very effective on gravel bases, but frog-rammers and heavy vibrating plates are particularly suitable on bases of crushed rock.

Summarizing remarks.—In wet climates, mechanically stable soils are not particularly suitable for constructing permanent airfields and they are quite useless for temporary military airfields. Where the climate is sufficiently dry to encourage their use, they are generally sufficiently stable to act as bases under a bituminous surfacing 4 inches thick if the tire

pressures of the aircraft to use the airfields will not exceed 100 lb. per square inch. Where higher tire pressures are likely to occur, the mechanically stable soil is suitable only as a sub-base under a base of greater stability.

Bases of soft crushed rock are eminently suitable for use in permanent airfields, since they have C.B.R. values ranging from 100 to 200 per cent. The effort to import the material is too great to permit their general use for temporary military airfields.

Examples of the use of mechanically stable materials in airfield construction are given in the Appendix (Tables 2 and 3).

Soil Stabilization with Cement

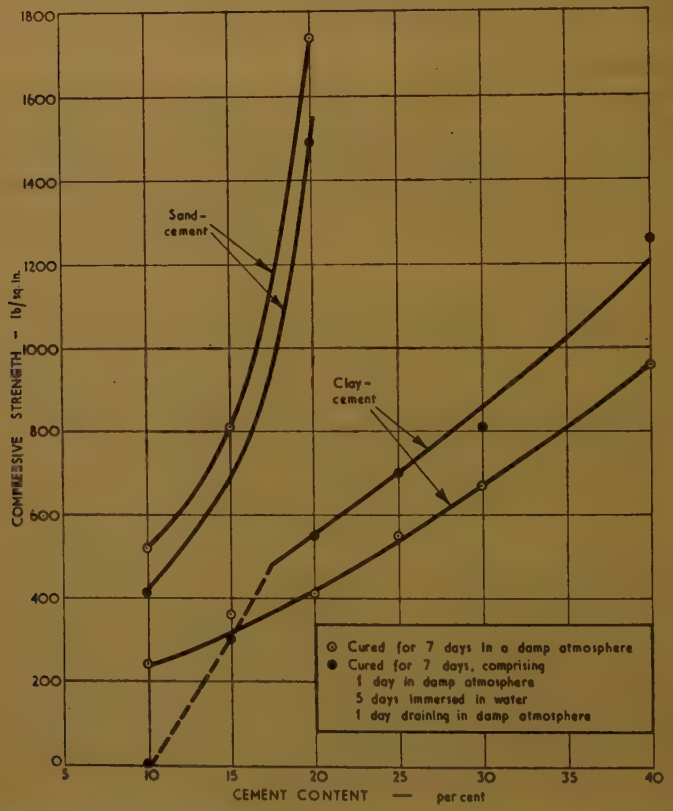
Soil stabilization with cement comprises all processes in which a stable material is obtained by using Portland cement as the main additive to the soil. The proportions of cement added to the soil usually range between 5 and 15 per cent by weight of the soil. Secondary additives are sometimes required to produce a stabilized soil with the required characteristics; these are usually employed in proportions of 1 to 2 per cent by weight of the soil.

When mixed with soil, Portland cement usually hydrates normally to form what may be regarded as a large number of rigid bonds in the soil. In soils which contain a certain amount of clay it is not possible, with the normal proportion of cement used, to do more than bond aggregations of the particles so that the stabilized soil comprises a combination of rigid cement bonds, which are unaffected by water, and of plastic soil bonds, whose strength is reduced by an increase in their moisture content. With clean granular soils a cement bond is usually formed between individual particles, so that the stabilized soil is more rigid than cement-stabilized clay soils and is comparatively unaffected by water. Thus, the unconfined compressive strength of soils stabilized with 10 per cent of normal Portland cement may range at an age of 7 days from about 1,000 lb. per square inch with well-graded granular soils to less than 200 lb. per square inch with some heavy clay soils, when they are cured at a constant moisture content. If the stabilized soils are immersed in water, the corresponding strength of stabilized granular soils is slightly affected, but a more pronounced reduction in strength is usually obtained in the case of stabilized clay soils (*Fig. 8*).

In the case of some granular soils they may be sufficiently mechanically stable in a dry and well-compacted state to provide a satisfactory base without further treatment, but where there is a likelihood of the soil becoming wet during or subsequent to construction, it is advantageous to stabilize it with cement. The resulting strength of the material may, however, be excessively high and it is considered that strengths in excess of 500 lb. per square inch at an age of 7 days should be avoided, since stronger materials will for reasons mentioned earlier in this Paper be liable to form more widely spaced and therefore more open cracks which do not permit

adequate interlocking and therefore result in a seriously defective structure
It may be possible to keep the strength of the material within the prescribe
limits by reducing the cement content to about 5 per cent or even less
With some granular soils, however, there may be a danger in employing
small cement content, since complete and satisfactory bonding may not b
achieved. In such circumstances it may be preferable to add 2 per cent o

Fig. 8

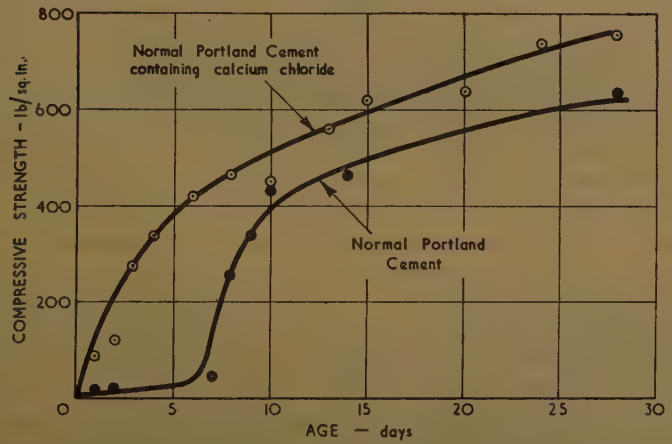


THE EFFECT OF CEMENT CONTENT ON THE COMPRESSIVE STRENGTH OF SPECIMENS
CLAY-CEMENT AND SAND-CEMENT, SHOWING THE EFFECT OF IMMERSING T
SPECIMENS IN WATER

a bitumen or tar emulsion to the soil stabilized with 8 per cent of cemen
The effect of the emulsion is to provide some plastic as well as rigid bonds
the stabilized soil. An alternative method of reducing the rigidity
cement-stabilized granular soils is to incorporate a small proportion
clay before adding the cement, but this may complicate unduly the practic
application of the process.

It has been found that some sands occurring in the United Kingdom, Europe, and Africa contain a very active form of organic matter which has the effect of delaying the hydration of normal Portland cement. Depending on the activity of this organic matter the delay may range from a few days to an indefinitely long period.[7] In order to induce normal hydration a special type of cement containing 2 per cent of calcium chloride by weight may be used with some soils. These effects are illustrated in *Fig. 9*. When the organic contamination is particularly large, proportions of calcium chloride up to 2 per cent by weight of the soil may have to be added separately. As with concrete, the use of calcium chloride accelerates

Fig. 9



THE RELATIONSHIPS BETWEEN COMPRESSIVE STRENGTH AND AGE FOR SPECIMENS OF AN ORGANIC SAND STABILIZED WITH NORMAL PORTLAND AND SPECIAL CEMENTS, SHOWING THE RETARDATION DUE TO ORGANIC MATTER WITH THE FORMER

the rate of hardening of all soils (including organic-free soils) stabilized with Portland cement.

In the stabilization of clay soils with cement it is usually necessary to produce a material which has an adequate stability when saturated with water. The cement-content employed in such cases must therefore be chosen to meet this requirement. An improvement in the properties of the stabilized soil in this respect can sometimes be achieved by the addition of about 2 per cent of hydrated lime.[8] This also has the effect of reducing the plasticity of the clay and making the soil more amenable to mixing with the cement.

It has already been pointed out that soil-cement with a flexural strength within the desirable range acts as a flexible material in which fine cracks form when it is subjected to traffic stresses. Since the sides of these cracks remain interlocked, the stability of the material is not adversely

affected, and there is in fact a close similarity in behaviour between soil-cement and a granular base composed of, say, crushed limestone.

In making laboratory measurements of the strength of stabilized soil, either by the C.B.R. or the unconfined compressive strength test, it is therefore considered that the effect of the cracking can be ignored. It is, however, necessary to consider the effect of the difference in efficiency of mixing obtained in practice and in the laboratory. Experiments have shown that, on an average, the strength obtained in the field with soil-cement at an age of 7 days is 60 to 70 per cent of laboratory-mixed soil-cement of the same composition and age. The criteria for the strength of stabilized soil required in the field must thus be multiplied by an appropriate factor to arrive at the corresponding laboratory strength that should be achieved by the material.

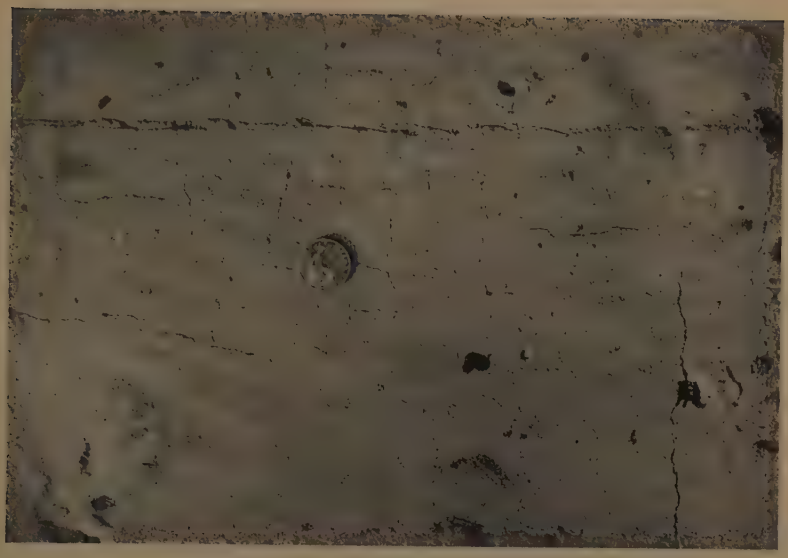
A further correction must be made to allow for the time which is expected to elapse in practice between the mixing of the soil-cement and its use by aircraft, since the strength of the material increases with age. It may be necessary to put temporary military airfields into use within a day of completion, whereas in the case of permanent airfields a period of several months may elapse. Thus, to allow for this, a higher 7-day laboratory strength is required for the military airfields and a lower strength for the permanent airfields. In certain circumstances it may be necessary to allow for the difference between the temperature of curing in the field and that in the laboratory, since temperature affects the rate of increase of strength.

Practical application of soil-cement.—It is not proposed to describe in detail the plant and methods for constructing soil-cement bases, since these have been adequately dealt with elsewhere.[3] There are, however, a number of matters which are particularly relevant to the use of the process for airfield construction.

Of the different methods of construction, that employing travelling mixers is considered to be the most suitable for permanent airfields, since it not only has a high rate of output but is suitable for laying considerable thicknesses of base. Mix-in-place is capable of high rates of output, but it is usually limited to laying the maximum thickness obtainable in a single layer (6 to 8 inches). The problem of laying and compacting several layers by this method of construction has yet to be solved satisfactorily. In the case of temporary airfields for military purposes, the advantages are thought to lie with the mix-in-place method, since thicknesses exceeding 6 inches are not usually required and since it also avoids the double handling of the soil.

Heavy clay soils present a special problem on account of the difficulty of mixing cement thoroughly into the soil. Experience has shown that no existing mix-in-place plant is particularly satisfactory for the purpose, since to obtain adequate mixing very many passes of the mixer are required even at the most suitable moisture content. At high moisture contents

Fig. 1



FORMATION OF CLOSELY SPACED FINE CRACKS IN CEMENT-STABILIZED BASE 5 INCHES THICK

Fig. 14

EXPERIMENTAL ROAD WITH BASES CONSTRUCTED WITH A UNIFORM SAND STABILIZED BY DIFFERENT PROCESSES AND COVERED WITH PREFABRICATED BITUMINIZED SURFACING (ON LEFT) AND WITH PRE-MIX BITUMINOUS SURFACING (ON RIGHT)

Fig. 15

MACHINE USED FOR SIMULATING ROAD TRAFFIC

Fig. 16

EXPERIMENTAL ROAD WITH BASES CONSTRUCTED WITH A HEAVY CLAY AND AN ORGANIC CLAY STABILIZED WITH CEMENT

Fig. 17

MACHINE USED FOR SIMULATING AIRFIELD TRAFFIC

Fig. 18

CRACKS OCCURRING IN A 6-INCH-THICK BASE OF CEMENT-STABILIZED SAND AFTER 200 COVERAGES OF A 25,000-LB. WHEEL LOAD AT 110 LB. PER SQUARE INCH. THIS PAVEMENT DID NOT GIVE INDICATION OF HAVING FAILED

Fig. 19

Cracks occurring in a 6-inch-thick Base of Cement-Stabilized Sand after 50 Coverages of a 25,000-lb. Wheel Load at 160 lb. per square inch. This Pavement was considered to have failed

Fig. 20

Prototype Single-Pass Mix-in-Place Machine

mixing is impossible. Investigations[8] have shown that very efficient mixing can be obtained with pottery pug-mills, and plant incorporating a pug-mill is in course of development.

Summarizing remarks.—It is considered that for all forms of airfield construction the use of cement provides the most satisfactory method of stabilizing soils that is at present available. Table 4 in the Appendix lists some of the airfields which have been constructed by this process. An important advantage is that it has a positive strengthening effect and thus gives ample stability with many soils. By the use of secondary additives and with the introduction of improved types of mixing plant it will be possible to treat successfully a very wide range of soil types. The process is of particular advantage in wet climates since the cement absorbs water during hydration; nevertheless, considerable success has attended its application in dry climates as is shown by the evidence given in the Appendix (Table 4) of airfields constructed in dry parts of the United States. Soil-cement may be liable to damage when exposed directly to alternate freezing and thawing or alternate wetting and drying conditions, but is unlikely except in the most severe climates to be affected under a substantial bituminous surfacing. The soil-cement process is considered to be particularly effective for strengthening the gravels occurring in the British Isles and in Western Europe, that in their natural state are unstable in wet conditions.

Soil Stabilization with Bitumens

There are two principal forms of stabilizing soils with bitumens, the object in one form being to obtain a material with a higher stability than that of the soil itself and in the second to increase the resistance of soil to the absorption of water and so assist in the preservation of the natural stability it possesses when in a dry and well-compacted condition.

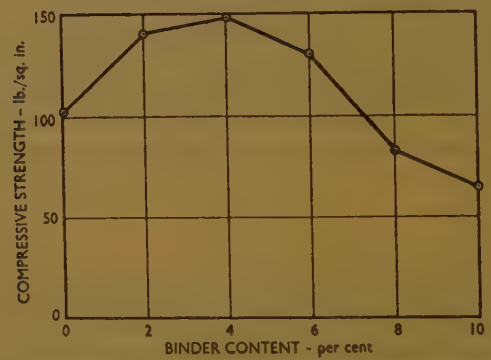
It has been found both by laboratory investigations and by experience in the field that bitumens are most successful in providing increased stability in the case of clean granular soils such as gravels and sands. These soils lack cohesion and in their natural state depend for their stability on the internal friction between particles that is developed as a result of a normal pressure on the plane of shear. It is a common experience that uniformly graded gravels and sands lack stability under a loaded wheel owing to relative movements occurring between individual particles. This is the practical result of the shear stresses in the soils near the surface exceeding the shear strength. The increased resistance to shear which is needed in such soils to enable them to function as a base material is imparted by bitumens, in the form of increased cohesion between the particles. Examples of the successful application of this principle are the dry bitumen-sand mix process used in the Middle East and in the United States, and the wet bitumen-sand mix process which was used on a considerable scale

during the 1939–1945 war to construct airfields in the United Kingdom (see Table 5 in the Appendix).

In these processes 4 to 10 per cent of a cut-back bitumen is usually employed, the bitumen being either of the rapid- or medium-curing type, that is, the solvent in the bitumen is either of high or medium volatility. In the wet bitumen-sand mix process the water is displaced from the surface of the sand particle by the bitumen, and excess water in the pores of the sand is driven out during the compaction of the stabilized material; in order that the bitumen shall displace the water it must contain an acid constituent and 1 or 2 per cent of hydrated lime is added to the sand before mixing.[9]

In the case of clayey soils the provision of increased stability by the addition of bitumens is much more difficult for two reasons. First, it is

Fig. 10



Relationship between Unconfined Compressive Strength and Binder Content for a Specimen of a Sandy Clay containing Cut-Back Bitumen

very difficult to mix any stabilizer satisfactorily with clayey soils and, secondly, in wet climates the total fluids content of bitumen plus water is usually so high that the stability of the treated soil is less than that of the natural soil. Generally, the function of bitumens in clayey soils would appear to be to replace the cohesion imparted by the pore-water by the higher cohesions of films of bitumen. Even in circumstances where the bitumen is used to best advantage, laboratory studies indicate that the gain in strength is not particularly great (*Fig. 10*). For these reasons it is not considered that the use of bitumens for strengthening clayey soils has considerable potentialities either in road or airfield construction.

When used to waterproof clayey soils, bitumens are added in comparatively small proportions of the order of 2 to 4 per cent. The increased resistance of soils to the absorption of water is believed to be caused by the formation of a film of bitumen on the surface of water in the pores of the

soil. The entry of water into the soil is hindered by the resistance of this bitumen film to displacement. Paraffin wax added to the bitumen helps to form films of greater rigidity, and waxed bitumens have been marketed especially for the waterproofing of soils. These are usually employed with 1 or 2 per cent of hydrated lime, whose function is to flocculate the clay particles in the soil, and thus bring the soil to a condition more suitable for the effective use of the bitumen.

When bitumens are used to waterproof soils, the aim is to treat the soils when they are in a dry and stable condition. When this is done the resulting C.B.R. of the soil has been found rarely to exceed 30 per cent, which is a suitable value for the sub-base of a permanent airfield but unsuitable for temporary military airfields.

It thus appears that apart from the bitumen-sand mix processes there is no other form of soil stabilization process with bitumens that is likely to offer a generally successful practical method in airfield construction.

In the construction of airfields by the wet bitumen-sand mix process during the war, a bitumen-stabilized layer 4 inches thick served the combined function of surfacing and base on the sand subgrade. With the increase in the wheel loads and tire pressure of aircraft it seems probable that for both permanent and temporary military airfields this material would now have to be used as a base with a surfacing of greater shear strength; for permanent airfields this would be a bituminous surfacing 4 to 6 inches thick. The introduction of jet-powered aircraft will also affect the future use of wet bitumen-sand mix as a surfacing for airfields.

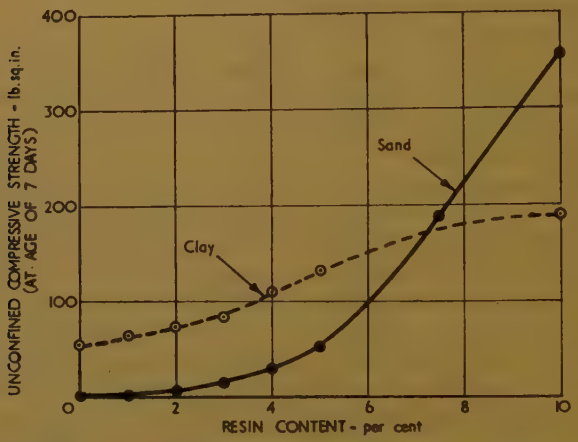
Soil Stabilization with Chemicals

In recent years considerable attention has been given both in the United Kingdom and in the United States to the possibility of using chemicals to stabilize soils, particularly in connexion with the construction of military airfields. The object of this work has been to produce a stabilizer that could be employed in lower concentrations than Portland cement or bitumens; that would become effective more rapidly than the latter; and whose performance would be less affected by the composition of the soil. The work has been concentrated on the types of organic chemical compounds that can be made to polymerize in situ in the soil after mixing, forming bonds between adjacent particles. The nature of these bonds and hence the mechanical properties of the resulting stabilized soil are influenced by the structure of the chemical employed. Thus, calcium acrylate, which has been examined by Lambe [10] in the United States, polymerizes when added to wet soil to give a rubber-like material, whilst resorcinol-formaldehyde, which has been studied in the United Kingdom, gives a hardened material similar to soil-cement.[11] A comprehensive summary of the different types of chemical stabilizer and an assessment of their relevant values has been given by Mainfort.[12]

As a result of laboratory work both in the United Kingdom and the

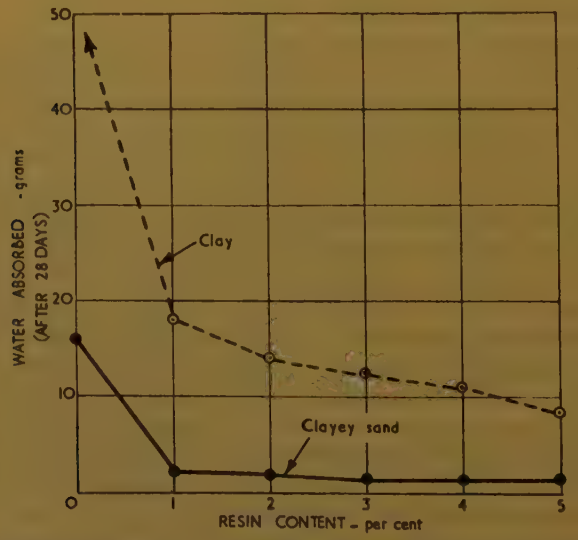
United States, it has been found that the proportions of chemicals needed to produce a given stabilizing effect are of the same order (10 per cent) as those required with normal Portland cement or bitumens (*Fig. 11*). In some cases, however, a waterproofing effect is obtained (*Fig. 12*), which

Fig. 11



Effect of Resin Content on Unconfined Compressive Strength of Sand and Clay Specimens stabilized with Resorcinol-Formaldehyde Resin

Fig. 12



Effect of Resin Content on Water Absorption of Clayey Sand and Clay Specimens stabilized with Resorcinol-Formaldehyde Resin

suggests that these chemicals might be used in small proportions as secondary additives with the more conventional stabilizers.

In view of the proportions of chemicals needed when materials of this group are used as primary stabilizing agents, it is unlikely that they will ever be applied to the construction of civil airfields, since the cost per unit weight is between thirty and one hundred times that of hydraulic cements or bitumens. In the case of military airfields, they enable no reduction to be made in the weight of material that must be transported to the site, and it is also probable that the chemicals involved may be required for other urgent purposes. Examination of their technical characteristics reveals that they possess no properties that can outweigh these factors, and there does not therefore appear to be any important grounds on which to justify their consideration for military purposes.

Full-Scale Trials of Soil Stabilization Processes

Since 1947, full-scale experimental lengths of stabilized soil have been constructed at five different sites, with the object of obtaining practical information about the performance of the material in relation to the requirements for temporary military roads and airfields.

The first investigations were carried out at two sites in connexion with the road problem, and were concerned primarily with comparing the relative merits of the different methods of stabilizing soils, whilst the investigations at the three remaining sites were carried out to study the performance, for use on airfields, of soil-cement which had been shown by the results of the earlier trials to be particularly suitable for the purpose.

Road Trials

The road trials were carried out with three soils: a uniformly graded sand; a heavy clay; and a highly organic clay. The particle-size distribution curve of the sand (Casagrande classification SU) is shown in *Fig. 13*, and the results of plasticity tests and Casagrande classification of the two clay soils are given in Table 1.

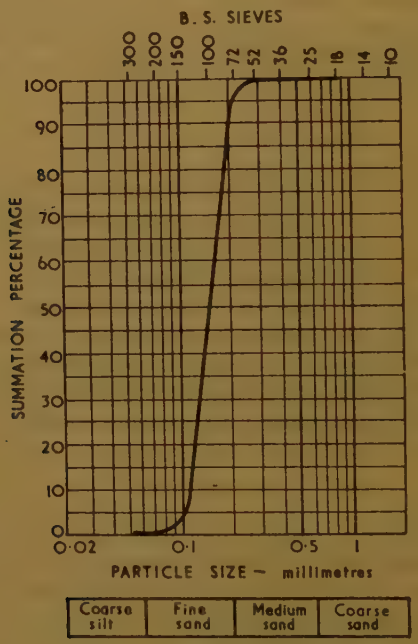
Table 1.—clay soils used in road trials

	Liquid limit: per cent	Plastic limit: per cent	Casagrande classification
Heavy clay	70	23	CH
Organic clay	73	44	OH

Laboratory tests were first made with these soils treated with a wide range of stabilizers. These showed that the chemical stabilizers were either ineffective, or, where effective, were required in proportions too large in relation to the supply position to justify their trial on a full scale.

At the sand site, the two principal stabilizers selected for trial on the basis of the results of the laboratory tests were cement and a special cut-back bitumen of low viscosity. Cement in the proportion of 10 per cent gave a compressive strength of about 300 lb. per square inch and a C.B.R. of about 400 per cent at an age of 7 days, whilst 6 per cent of the bitumen with 2 per cent of a hydrated lime gave a C.B.R. of 20 per cent at 7 days. This was in effect the wet bitumen-sand mix process applied with the standard bitumen used by the Army.

Fig. 13



TYPICAL GRADING CURVE OF THE LYNDHURST SAND

Sections were laid to thicknesses of 4 inches and 7 inches, and half the width of the sections was covered with a prefabricated bituminized surfacing and the other half by a dense gravel bituminous surfacing $1\frac{1}{4}$ inch thick (*Fig. 14**). The experimental road was subjected to traffic with wheel loads ranging from $1\frac{1}{2}$ to $7\frac{1}{4}$ tons at tire pressures from 50 to 80 lb. per square inch. A total of 16,000 axles used the road intermittently over a period of 2 years. In addition, a traffic simulator (*Fig. 15*) was used to cover specific areas of the experimental sections with 100,000 passes of a wheel loaded to $1\frac{1}{2}$ ton at a tire pressure of 50 lb. per square inch.

The trial showed that under the prefabricated surfacing the soil-cement

* *Figs 14–20* are all photographs and will be found between pp. 466 and 467.

base had satisfactory stability with a 7-day field strength of 310 lb. per square inch, but was unsatisfactory with a 7-day field strength of 170 lb. per square inch. The bitumen-stabilized bases were seriously deformed when covered with the prefabricated surfacing. All the experimental sections behaved more satisfactorily under the 1¼-inch bituminous surfacing than under the prefabricated bituminized surfacing; more deformation was experienced with the bitumen-stabilized base than with the cement-stabilized base.

The main conclusions to be drawn from these trials are that :—

(1) The criterion of 250 lb. per square inch at 7 days was satisfactory for soil-cement bases for temporary military roads with a negligible thickness of surfacing.

(2) The stabilized soil bases had a much improved performance when covered with a thin bituminous surfacing.

(3) On the uniform sand a satisfactory road for military purposes was obtained with soil-cement and soil-bitumen bases covered with a bituminous surfacing 1¼ inch thick, or with a soil-cement base covered only with a prefabricated bituminized surfacing.

Laboratory tests with the heavy clay showed that only cement was satisfactory for stabilizing the soil, and a laboratory strength of 250 lb. per square inch at 7 days was obtained with 15 per cent of Portland cement and 2 per cent of hydrated lime. Using a pottery pug-mill for mixing the clay and the stabilizers, experimental sections have been successfully laid for traffic testing. Provided satisfactory curing is established within ¼ hour of compaction, the stabilized clay is very resistant to all kinds of weather. This work has been described in detail elsewhere.[8]

Laboratory tests followed by field trials have shown that, even with 25 per cent of cement, it is possible to obtain with the organic clay a field strength at 7 days of only 100 lb. per square inch, and it is considered that this soil is beyond the limit of those which it is worth while stabilizing. However, an experimental section of cement-stabilized organic clay has been included at the experimental site where the bases with cement-stabilized heavy clay have been constructed (*Fig. 16*).

Although traffic trials have still to be carried out on the stabilized heavy clay, it is considered that the road trials generally have demonstrated that cement can be used in economic proportions to stabilize most soils to produce a satisfactory base material for roads. The essential requirement is to obtain adequate mixing of the materials and to compact the treated soil adequately when laid. With the heavy clay soils the pug-mill seems to be the most efficient type of mixer, whilst impact methods have provided the most effective compaction.

Airfield Trials

The airfield trials are being carried out on a range of granular and cohesive soils with different bearing capacities. The construction and

testing of experimental sections have so far been undertaken at three sites where the soils are respectively a uniform sand, a sandy gravel, and a clay. The performance is being compared of a number of types of construction suitable for forward military airfields, and amongst these is a base of cement-stabilized soil—unsurfaced or covered either with a prefabricated bituminized hessian or metal surfacing. The bases were constructed to different thicknesses estimated by means of the C.B.R. method.

The experimental sections are being subjected to the equivalent of aircraft traffic with a special trolley designed by the Royal Aircraft Establishment (*Fig. 17*). With this trolley, wheel loads up to 50,000 lb. with tire pressures up to 160 lb. per square inch can be applied to the different constructions. In the tests made so far, about 200 coverages have been applied to each experimental section, one coverage being a number of passes of the trolley so arranged that a strip about 10 feet wide is tested with the path traversed in each pass immediately adjacent to the path followed in the previous pass. The performance of the sections is judged mainly by the increase in the settlement of the surface with number of coverages of the loaded wheel, resulting from deformations in the subgrade and in the base.

The results obtained to date can be summarized as follows :—

(1) On the two granular soils a cement-stabilized base 6 inches thick was adequate for carrying the 200 coverages of a wheel loaded to 50,000 lb. and a tire pressure of 110 lb. per square inch.

(2) Cement-stabilized granular soil with a 7-day field strength of 160 lb. per square inch was strong enough to withstand the stresses directly imposed by a pneumatic-tired wheel having an inflation pressure of 160 lb. per square inch, but a specially designed weak material with a 7-day field strength of 112 lb. per square inch failed.

The 7-day field strength of 200 lb. per square inch suggested earlier is thus confirmed as a reasonable criterion for soil-cement bases for forward airfields.

It was observed that the nature of the cracking of the soil-cement was related to its performance. This is illustrated by *Fig. 18*, which shows the cracking produced in the soil-cement by 200 coverages of a wheel loaded to 25,000 lb. at 110 lb. per square inch, and by *Fig. 19*, which shows the cracking produced in the same material by 50 coverages of the same wheel load of 25,000 lb. but with a tire pressure of 160 lb. per square inch. (To record the cracks vegetable oil was used to accentuate them.) In the case of the soil-cement shown in *Fig. 18*, no failure occurred, and the fine cracks formed at the spacing seen in the Figure after a few coverages and then remained static. The cracking seen in *Fig. 19* was progressive, additional cracks continually forming until a state of failure was reached.

Fig. 21

Arrangement of plant for stabilizing Sand with Cement, using Modified Bitumen Mixers

Fig. 22

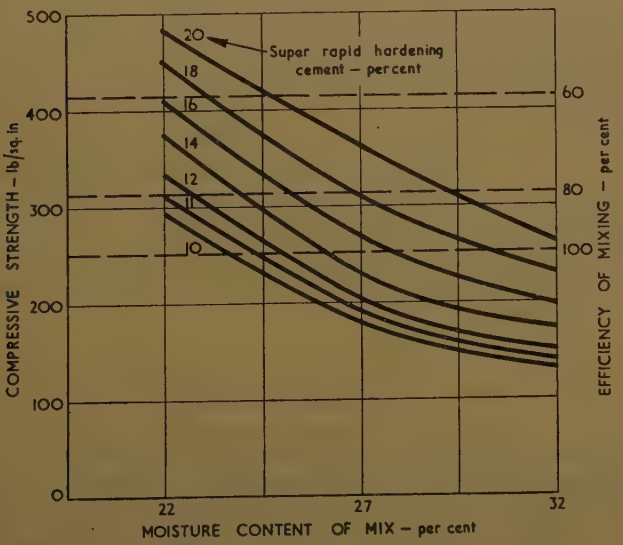
Pottery Pug-Mill used for mixing Cement with Clays

Plant Performance Trials

In the course of the road and airfield trials, experimental studies have been made of the quality of mixing of soil and cement obtained with different types of machine. The efficiency of mixing was measured in terms of the percentage of the laboratory compressive strength obtained with the field mixers. On the whole, stationary mixers of either the batch or continuous-feed type have been found to possess higher efficiencies than rotary tillers used in the mix-in-place method. It is considered that there is room for considerable improvement in the design of these machines and studies of the effect of the speed of the rotor in relation to the forward speed of the machine and of the effect of changes in the type of tine used by the rotor are in hand. The aim is to produce a machine which has a high efficiency with only a single pass over the ground. A prototype machine that has been developed is shown in *Fig. 20*.

With regard to stationary plant, asphalt mixers have been modified (*Fig. 21*) to give a continuous process with high efficiency and an output of 20 tons per hour under favourable conditions. A high efficiency of mixing with heavy clay soils was obtained with a batch-fed pug-mill (*Fig. 22*) of the type used in the pottery industry, with which outputs of the order of 10 tons per hour were achieved.

Fig. 23



Effect of the Efficiency of Mixing on Cement Content to obtain a Required Compressive Strength of Soil-Cement

The importance of the investigations aiming to produce higher efficiencies of mixing is illustrated by *Fig. 23*, which shows how an increase of

20 per cent in the mixing efficiency can enable a saving of one-fifth of the cement to be achieved. In this instance, the specification required a strength of 250 lb. per square inch in the field.

Conclusions

In view of the considerable saving of cost and materials that can be achieved, there is a strong case for the use of soil-stabilization processes for constructing the bases of airfields in cases where it can be demonstrated that the material produced is satisfactory for the purpose. Many airfields constructed in various parts of the world have proved that stabilized soil has the necessary strength and durability when covered with a suitable surfacing.

Of the various processes, soil-cement has the greatest potentialities, since it can be successfully applied to a wide range of soils and climatic conditions. The complete mechanization of the process not only enables high outputs to be achieved but also renders the process no more susceptible to weather during construction than any of the orthodox methods. Research has shown that it is able successfully to withstand the direct application of wheel loads of 50,000 lb. and tire pressures of 160 lb. per square inch; this would be adequate for temporary military airfields.

Bitumen stabilization has potentialities in airfield construction only in the case of relatively clean granular soils, but further studies are desirable to establish how far the bitumen-sand mix processes will meet the requirements which arise in modern airfield construction.

Mechanical stabilization is suitable for use in fairly dry climates, but in wet climates some types of so-called mechanically-stable soils are very unstable. The most stable materials are those in which the particles are partly crushed in the compaction process. The large volumes of material that have to be imported preclude the use generally of mechanically stabilized materials for temporary military airfields.

Research on chemical methods of soil stabilization has indicated that there is unlikely to be a case for the consideration of these methods in the foreseeable future.

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Further references are given on p. 485.

The Paper is accompanied by ten photographs and twelve sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

A Survey of the Use of Stabilized Soil in Existing Airfields

A survey has been made of published references and information at the Road Research Laboratory and elsewhere, relating to the use of stabilized soils and soft aggregates in the construction of existing airfields. It is not possible to list in detail all the instances where this form of construction has been employed, so that the information presented has been selected to give a representative picture of what has been done (Tables 2 to 5).

Most of the airfields listed were constructed during World War II, which was a period of considerable activity in this field, and were designed chiefly for military purposes. Subsequently, many were used for civil flying and with the advent of heavier aircraft have been built up, usually by the construction of additional bituminous surfacings.

Stabilized soil has been most widely used in the United States, where the area

TABLE 2.—AIRFIELDS WHERE MECHANICALLY STABILIZED SOIL BASES HAVE BEEN USED

Country	Location	Base material	C.B.R. of base material (in place) : per cent	Thickness of base : inches	Thickness of surfacing* : inches	Reference
Australia . .	Canberra	Shale and loam soil		14		(13)
Canada . .	Le Pas, Manitoba	Gravel		6	3	(14)
,,	Yarmouth, Nova Scotia	,,		6	3	(15)
,,	A number of flight strips on the Alaska Highway	,,				(16)
,,	Mingan, Quebec	Crushed rock		8	$2\frac{1}{2}$	(17)
Cook Islands .	Raratonga	Coral-clay mixture				(18)
Colombia . .	Calpuerto, Cali	Sand-clay		10	$1\frac{1}{2}$	(19)
Finland . .	Various	Gravel-soil mixtures		2·4		(20)
France . .	Corsica	Gravel-clay				(21)
Hong Kong .	Pingshan	Beach sand and decomposed granite gravel		6	Stone macadam + 2″	(22)
Morocco . .	Targa	Sandy gravel and silty soil				(23)
,,	Salé	Gravel-clay		6	2	(24)
,,	Rabat	,,		4–5	Triple surface dressing	(24)
Portugal . .	Lisbon	Stone pitching and crushed stone		10	2	(25)
United States	Washington National Airport	Gravel		9	Rolled asphalt	(26)
,, ,,	Idaho Falls, Idaho	Crushed gravel				(27)
,, ,,	North Bend, Oregon	Gravel				(28)
,, ,,	,, ,, Virginia	Gravel-sand-clay		8	$1\frac{3}{4}$	(29)

TABLE 2.—*continued*

Country	Location	Base material	C.B.R. of base material (in place): per cent	Thickness of base: inches	Thickness of surfacing*: inches	Reference
United States	Albuquerque, N. Mexico	Gravel	37–175 +	9	2	(30)
,, ,,	Bangor, Maine	,,	80 +	$26\frac{1}{2}$	$3\frac{1}{2}$	,,
,, ,,	Lewistown, Montana	,,	28	6	6	,,
,, ,,	Pueblo, Colorado	,,	75–100 +	$9\frac{1}{2}$	3	,,
,, ,,	Yuma, Arizona	,,	42–62	$4\frac{1}{2}$–10	$2\frac{1}{2}$–$5\frac{1}{2}$	,,
,, ,,	Boise, Idaho	,,	80 +	$12\frac{1}{2}$	2	,,
,, ,,	Casper, Wyoming	Sand	—	8	6	,,
,, ,,	Clarksville, Tennessee	Gravel-sand-clay	48–100	9	2	,,
,, ,,	Minden, Nevada	Gravel	—	$4\frac{1}{2}$–16		,,
,, ,,	Salt Lake City, Utah	,,	14	4	7	,,
,, ,,	Smyrna, Tennessee	,,	80 +	11	5	,,
,, ,,	Woodward, Oklahoma	,,	28	9	3	,,
,, ,,	Tucson, Arizona	,,	—	8	4	,,

* Usually hot-rolled asphalt.

TABLE 4.—AIRFIELDS WHERE CEMENT-STABILIZED BASES HAVE BEEN USED

Country	Location	Area : square yards	Soil	Cement content : per cent	Thickness of base : inches	Surfacing	Reference
United Kingdom	Greenham Common, Berks.	Dispersal arm 1,000	Gravel	15	6	Two-course bituminous carpet	
,, ,,	Aldermaston, Berks.	Perimeter track 10,000	Gravel	10	6	Two-course bituminous carpet	
,, ,,	Honington, Suffolk	Perimeter track 120,000	Fine sand	7·5–15	6	Surface dressing	
,, ,,	Hatfield, Herts.	11,000	Sand	10	6	Concrete	(31)
,, ,,	Heathrow, Middlesex	Runways	Gravel		12	Concrete	
,, ,,	Fairford, Glouc.	Runway and perimeter track 170,000	Oolitic limestone gravel	7·5*		4-inch bituminous carpet	
,, ,,	Wisley, Surrey	Runway 180,000	Gravel and stone	10	8–9	Two-course bituminous carpet	
Australia . .	Bourke and Roto, New South Wales	Runways 350,000	Clayey sand	10*	6	Surface enrichment with cement	(32)
Canada . . .	Camp Borden	330,000	Wind-blown sand		6		(33)
,,	Seven Islands, Quebec	450,000			6		,,
Hong Kong .	Kai Tak	Taxiway 15,000	Decomposed granite gravel	12	5	1-inch sand-bitumen carpet	(34)
Germany . .	Several military airfields	Runways	Sandy soils				(35)
France . . .	Limoges	Runway 34,000		10–11			
Norway . . .	Two military airfields	Runways 80,000	Gravel	12	6		(36)

TABLE 4—*continued*

Country	Location	Area : square yards	Soil	Cement content : per cent	Thickness of base : inches	Surfacing	Reference
United States	Columbus, Nebraska	37,000	Gravel-sand-clay	7*		2-inch bituminous carpet	(37)
,, ,,	Philadelphia, Penn.	Taxiway 20,000	Gravel-sand-clay	10–12	6	1½-inch bituminous carpet	(38)
,, ,,	Duluth, New York	Runways	Gravel			Bituminous carpet	(39)
,, ,,	Lubbock, Texas	Runways 119,000	Gravel and stone	7*	6–8	Penetration bitumen macadam	(40)
,, ,,	——, Texas	Runways 190,000		12*	6	1½-inch rock asphalt	(41)
,, ,,	Johnsville, Penn.	Runways 100,000		10*			(42)
,, ,,	Pasco, Washington	1,700,000		7			(43)
,, ,,	Alexandria, Louisiana	430,000					(33)
,, ,,	Avon Park, Florida	550,000					,,
,, ,,	Chicopee Falls, Mass.	430,000					,,
,, ,,	Columbia, South Carolina	640,000					,,
,, ,,	El Paso, Texas	750,000	Clayey sand	8–12*	usually 6	Usually 1–1½ inch hot-rolled asphalt	,,
,, ,,	Nursery, Texas	530,000					,,
,, ,,	Ottumwa, Iowa	840,000					,,
,, ,,	Peru, Indiana	590,000					,,
,, ,,	Savannah, Georgia	470,000					,,
,, ,,	Victorville, California	800,000					,,

* Cement content by volume

TABLE 5.—AIRFIELDS WHERE BITUMEN-STABILIZED SOILS HAVE BEEN USED

Country	Location	Construction and approximate area : sq. yd.	Soil	Additive and content : per cent	Thickness : inches	Surfacing	Reference
United Kingdom	Squire's Gate, Lancs.	250,000	Dune sand	Special cut-back bitumen—7·5*	4		
,, ,,	Valley, Anglesey	250,000	Dune sand	Special cut-back bitumen—7·5*	4	Open-textured bituminous carpet	
,, ,,	Woodvale, Lancs.	240,000	Dune sand	Special cut-back bitumen—6·5*	4	Open-textured bituminous carpet	
,, ,,	Benbecula, Inverness	220,000	Calcareous dune sand	Special cut-back bitumen—6·5*	4		
,, ,,	Tiree, Argyllshire	220,000	Calcareous dune sand	Special cut-back bitumen—8·5*	4		
,, ,,	Islay, Argyllshire		Calcareous dune sand				
,, ,,	North Connel, Argyllshire	52,000	Gravelly beach sand	Special cut-back bitumen—5·5*	4		
,, ,,	Carnaby, Yorks.	750,000	Beach sand	Special cut-back bitumen—4·5*	4		
,, ,,	Woodbridge, Suffolk	1,000,000	Pit sand	Special cut-back bitumen—5·5*	4	Sand surface-dressing	
,, ,,	Angle, Pembroke	Perimeter track 50,000	Dune sand	Special cut-back bitumen—6·5*	4	Bituminous carpet	
,, ,,	Waddington, Lincs.	Perimeter track 80,000	Clayey sand	Special bituminous oil—3·0 + hydrated lime—2·0			
United States	Gustavs, S.E. Alaska		Sand	Bituminous emulsion. Also bitumen-sand mixes + "Kotal," $Ca(OH)_2$ and cement	5	2-inch wearing course sand-bitumen emulsion	(44)

TABLE 5.—*continued*

Country	Location	Construction and approximate area : sq. yd.	Soil	Additive and content : per cent	Thickness : inches	Surfacing	Reference
United States	Northway, Alaska		Sand	Bitumen emulsion	3	2-inch wearing course sand-bitumen emulsion	(45)
,, ,,	Naknek, Bering Sea		Sand	Bitumen emulsion			(46)
,, ,,	Staten Island, N.Y.	Approx. 35,000	Sand	3·75–3·85 gall. sq. yd. sand-asphalt	7	Bitumen-emulsion and limestone chippings	(47)
,, ,,	Ballston Spa, N.Y.		Sand	8·5% Bitumen-emulsion	6	Bitumen and crushed limestone	(48)
,, ,,	Langley, Virginia		Sand	Bitumen emulsion		2–3 inch asphaltic concrete	(49)
,, ,,	Burlington, Vermont		Gravel sand and gravel sand + 100 lb./sq. yd. crushed stone	Bitumen emulsion	4½–6	3-inch cold plant-mix	(51)
,, ,,	Baton Rouge, Louisiana		Coarse sand and native sandy soil	Hot mix asphalt	3½	1½-inch sand asphalt	(52)
,, ,,	Corpus Christi, Texas	704,000	46% washed reef shell 46% flour buff sand	7½ bitumen emulsion + ½% lime	3½	5-inch single course cold mix. 55% shell, 15% sand, 22% flour buff sand, 8% emulsion	(53)

* With the addition of 2 per cent of hydrated lime.

constructed on airfields probably exceeds that in the rest of the world. Most of the published information is also of American origin.

Mechanical Stabilization (Table 2, pages 478–9)

This is probably the most widely used form of stabilized soil construction, and has been employed with success at some airports carrying very heavy traffic—for example, the Washington National Airport. Imported material of selected composition has usually been employed and the thickness of the base varied according to the bearing capacity of the subgrade. In most of the American examples quoted the construction has been designed for wheel loads of the order of 20,000 to 30,000 lb., and a survey [30] has shown that no failures occurred where the C.B.R. value of the soaked base material, as measured in the laboratory, was more than 40 per cent. It is also stated that, to prevent failure of the base, the plasticity index of the material has to be less than 5 per cent. It is noteworthy, however, that most of the examples quoted were constructed in areas where the climate is predominantly dry.

Soft Aggregates (Table 3)

Many airfields have been constructed in the southern states of the United States, using bases made with Caliche, a calcareous deposit resembling soft chalk, but use has also been made of such materials as ferruginous " pebble soil," shell, and slag. It is understood that use was also made of coral by the American armed forces in the Pacific Islands. In most of the cases quoted, the wheel loads carried are in the range 20,000 to 30,000 lb., and the thickness of base varied according to the supporting power of the subsoil, 10 inches being an average figure. Again, the use of these materials has been confined to dry climates.

TABLE 3.—AIRFIELDS IN THE SOUTHERN STATES OF THE U.S.A. WHERE BASES CONSTRUCTED WITH SOFT AGGREGATES HAVE BEEN USED [30]

Location	Base material	Thickness of base : inches	Thickness of surfacing* : inches
Albuquerque, N. Mexico	Caliche	9	2
Artesia, N. Mexico	Caliche-gravel	8	2
Clovis, N. Mexico	Caliche	12	1½
Las Cruces, N. Mexico	,,	9	1½
Washington, D.C.	Slag	4½	1½
Wink, Texas	Caliche	9–15	2
Amarillo, Texas	,,	10	1½
Gainesville, Texas	Crushed limestone	12	2
Hobbs, N. Mexico	Caliche	8	2
Roswell, N. Mexico	,,	11	2

* Usually hot-rolled asphalt.

Cement Stabilization (Table 4, pages 480–1)

In the United Kingdom, cement-stabilized imported gravels are quite frequently laid under concrete surfacings to strengthen the foundation which may often be clay, but the process is understood to be regarded mainly as a construction expedient (for example, in winter working).

In Germany, several airfields are understood to have been constructed with cement-stabilized soils, usually sands. The work was undertaken by the Germans in the closing months of World War II when airfield facilities were urgently needed immediately before and during the Allies' final advance. The mix-in-place method was used and several types of travelling machines were employed, similar in conception to the larger types of American machines. Most of these airfields are believed to have gone out of service during the Allied occupation, and sufficient information is not available to assess accurately the usefulness of these runways.

Soil-cement has been employed to a very considerable extent in the United States, where 23 million square yards of runways has been constructed by January 1952,

chiefly for aircraft with wheel loads up to 15,000 lb. Most of the bases were of the order of 6 inches thick, a value that has been dictated more by the limitations of existing construction plant than by design considerations. It is probable that this limitation of thickness of soil-cement base to 6 inches has delayed the application of the process to airfield construction, since if soil-cement is regarded as a flexible base and designed in the appropriate thicknesses, runways capable of carrying much heavier loads could undoubtedly be constructed successfully with it.

It is interesting to note that, in two of the cases quoted in Table 4, soil-cement construction was adopted as a method of salvaging existing mechanically-stabilized soil bases that had failed under traffic. At one of the sites, gravel of unsuitable composition had been employed, and this material was rendered stable after treatment with cement and mixing in the broken bituminous surfacing.

Bituminous Stabilization (Table 5, pages 482–3)

Construction of airfield runways by the wet bitumen-sand mix process was undertaken successfully during the war at several airfields in the United Kingdom. As in the United States, the soils concerned were all sandy and the subgrades were also composed of the same material, having a good supporting power. Stationary mixers were usually employed, although travelling mixers of the American Barber-Greene type were also successfully used. Most of these airfields are still in use, although in one or two cases a bituminous surfacing has been added to cater for heavier traffic.

An omission from Table 5 is the lack of reference to those airfields, constructed mainly prior to 1939 in the Middle East, using dry bitumen-sand mix. The method of construction employed was similar in principle to that employed with wet sandy soils in the United Kingdom, but the addition of hydrated lime necessary to promote good coating and adhesion in the latter method was omitted. Little has been published regarding the construction and performance of these airfields, but it is believed that they have given satisfactory service.

Appendix References

(References 1–12 are given on p. 477)

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Discussion

Mr G. S. Cooper, referring to the use of soil stabilization for permanent airfields, observed that most of the pavements constructed during the 1939–45 war had, for a large number of reasons, been constructed in concrete. There had since been little new construction until comparatively recently, and then construction had consisted in either strengthening existing pavements for aircraft of normal weight or constructing new pavements for heavy aircraft. Most of the sites for heavy aircraft had subgrades with extremely low California Bearing Ratios (of the order of 3 per cent) and on those sites about 4 feet construction thickness would have been required if flexible pavements were to be used. On such sites, rigid pavements were more economical, the thickness of concrete required being of the order of 12 inches. Using the Authors' own basis of comparison, where they had pointed out that the cost of soil-cement stabilization was, inch for inch, about one-third that of concrete construction, it was clear that the latter was cheaper. In any case, the problem of laying and compacting thicknesses of stabilized soil of the order of 3 to 4 feet had still to be solved satisfactorily, and there was not as yet sufficient evidence that it was possible to reduce the thickness of pavement required by the C.B.R. design curves when using soil-cement stabilization.

In recent Air Ministry work, on at least three sites with subgrades of high bearing value, soil-cement bases had been used and had been very satisfactory, and soil-cement was proposed on at least two further sites. On at least a further three sites, mechanically stable gravel, obtainable locally at very low cost, had been used and had also given satisfactory results. The Authors could rest assured, therefore, that the Air Ministry was well aware of the potentialities of soil stabilization for permanent airfield construction and made use of the method whenever a suitable opportunity occurred. It was normal practice to investigate all possible forms of construction and adopt the most economic which could be guaranteed to give satisfactory results.

The Authors had described mechanical stabilization as being quite useless for temporary airfields in wet climates. That was a somewhat sweeping statement which might require qualification. Mechanically stable bases ought to work if they were adequately protected from moisture changes. If that could be done, mechanical stabilization, using locally available material, should obviously be the first method to be considered where rapid construction was required. In the penultimate paragraph of the " Conclusions," the argument was used against mechanical stabilization that the transportation problem would often render it impracticable, but if it were possible to obtain mechanically stable material a mile or two away from the site of the construction work, it would be much easier to use that

material than to import 2,000 tons or so of cement or other material, perhaps by air, which would be an enormous task. The task might have to be faced, but some alternative, if at all possible, had to be considered in the first place.

The Authors had made a very interesting point about the strength of soil-cement bases. On a subgrade of high bearing strength it might be found that the thickness of flexible pavement required would be less than that required for a properly designed rigid pavement, so that if one were to lay a cement-stabilized base, which was stronger than necessary to perform properly as a flexible base, one would in effect have a weak under-designed rigid pavement, which would commence to crack under loading and show all the troubles and faults common to weak rigid pavements.

Referring to comments on pp. 454 and 455, concerning the strength of the base and the thickness of surfacing, Mr Cooper pointed out that not a great deal of experience had as yet been obtained with tire pressures of 250 lb. per square inch or more, but what experience was available tended to show that a 4-inch thickness of surfacing and C.B.R. values of the order of 100 per cent in the upper layers of the base were sufficient. He knew that there was no disputing the mathematical analysis which the Authors gave, but it might be that a surcharge or a cohesive effect was somehow involved, which prevented shear displacement, so that even though there was local over-stressing it did not result in any failure.

Under the heading "Plant Performance Trials," the Authors had referred to the development of suitable plant. What thickness, in the Authors' view, would that plant be able to stabilize with only a single pass, and how should greater thicknesses be dealt with? Would the Authors also give some guidance on the maximum clay fraction beyond which stabilization with cement was not economically practicable?

Mr W. P. Andrews observed that the Authors had stated on p. 449 that stabilized soil had in post-war road construction been cheaper by 4*d*. per square yard per inch thickness than hardcore. That seemed to require some qualification. Mr Andrews had been told in Lancashire, only two weeks ago, that nothing could compete with hardcore there, because it was obtained from slag-heaps. In the south, however, it was becoming more difficult to obtain and much more expensive, and probably the figure there would be even higher than 4*d*. per square yard.

A comparison was made on p. 451 between rigid and flexible pavements. He did not find that comparison very satisfactory, for the Authors had emphasized too strongly, he thought, the fact that cracks might occur in a rigid pavement and that those cracks might open to such an extent that the aggregate interlock might be spoilt. That took no account of the possibilities of prestressed concrete, in which the cracks remained permanently closed, or the use of ordinary reinforcement in the rigid pavement, which would prevent the cracks from opening.

The Paper was devoted to the stabilized-soil base under bituminous

surfacings. In one or two instances a soil-cement base had been used under concrete roads. It had made an excellent substitute for lean-mix concrete or a granular base. It had the great advantage, in a large job with central mixing, of providing a road over which the concrete could be transported to the site from the central mixing plant without being damaged. Experience in the past few months had shown the necessity for an absolutely impervious surfacing to the stabilized soil, and concrete would supply that.

Fig. 17 showed the special trolley used for simulating airfield traffic. He had seen that at work, and the results of those tests had been remarkable in showing how a 6-inch thickness of stabilized soil stood up to such severe punishment. He hoped that the work which was being done by the Military Engineering Experimental Establishment would show results, especially with regard to the machine shown in *Fig. 20*, which would be very much cheaper than an American single-pass stabilizer such as the P. & H. There was need for British plant for that work; Great Britain was far too dependent upon the United States. There was need for plant to pulverize the soil and for mixing the soil with the cement, and it was important to produce in Britain a cement spreader to replace spreading by hand, which had been shown to be inefficient. Better plant was needed to distribute moisture uniformly and at a controlled rate.

On the question of the testing of soils, in the United States great importance was attached to a freezing-and-thawing test and to a wetting-and-drying test—especially to the former. The freezing-and-thawing test had never been adopted in Britain, and during the past 3 months there had been several localized failures of soil-cement roads, and in every case the weaknesses had been disclosed by the first winter frost. By the use of a freezing-and-thawing test those small local failures might have been avoided. It would be interesting to have the Authors' views on freezing-and-thawing tests.

Mr G. C. Wilson remarked that he was not entirely satisfied that it was necessary to limit the strength of soil-cement for the purpose of controlling cracking, and that except for obvious reasons of economy, there was no justification for prescribing a maximum 7-day strength of about 500 lb. per square inch. He did not favour the idea of using extra additives to keep down the strength when, for practical reasons, it was necessary to use more cement than was actually required to give a strength value below that maximum.

He also found that the criteria for strength of soil-cement bases, given on pp. 456 and 466, were slightly confusing. On p. 456 the suggestion was made that, with the lower range of tire pressures, the 7-day strength for a permanent airfield should be 250 lb. per square inch, whilst for military airfields a minimum strength of 200 lb. per square inch would be adequate. On p. 466 it was stated that, owing to the possible necessity for putting military airfields into commission with the least possible delay, a higher 7-day strength was required for them than for permanent airfields. In

view of that apparent contradiction, he would like to ask if the value of 200 lb. per square inch was sufficiently high for military airfields.

With regard to the behaviour of crushed rock in mechanically stable base construction, although he agreed that materials, even quite soft ones, which crushed under the compacting plant could provide excellent bases, he found it difficult to accept the statement that hard rock structures were more susceptible to moisture-content changes than those of softer rock. It had been firmly established by laboratory tests that the angle of internal friction of hard granular material was independent of moisture content, and he would be very surprised if the hard rocks, when adequately restrained by a thick surfacing such as would be used on a permanent airfield, would prove in any way inferior to the softer ones; but perhaps the Authors could refute that suggestion from their own experience.

He had assumed that *Fig. 7* referred to a sandy gravel in which there was a proportion of clay, in which case the results shown could well be understood. Would the Authors confirm whether it was a clay gravel or a clean hard material?

Finally, on the question of mechanically stable bases, he did not think that the condemnation of gravel-sand-clays for use in wet climates was entirely warranted. As stated on p. 462, those materials were virtually impermeable after compaction and did not suffer unduly in the presence of water. Surely, then, it was the condition of the material when placed which was the controlling factor. Admittedly those materials could be difficult to handle in wet weather and were far from easy to dry out subsequent to becoming wet, but once properly compacted and covered with an impervious surfacing they should be capable of good service, even for temporary airfields.

Turning to the construction aspect of stabilization, Mr Wilson gave some details of the work carried out at Wisley, to which reference was made in Table 4 of the Appendix. So far as he was aware, that was the largest single contract for cement stabilization which had been completed in the United Kingdom. It had involved the improvement of an existing runway and taxiway pavement which consisted of a base of hoggin of 8 inches nominal thickness covered with a bitumen-macadam surfacing $2\frac{1}{2}$ inches thick. A few areas had been provided with an asphalt wearing-course. The description in Table 4 of the original base material as " gravel " was rather misleading. The mix-in-place method had been employed and the procedure had been to rip up the surfacing and base, break up the lumps of macadam by pulverizing with rotary machines until the coated stone had been evenly distributed throughout the hoggin, and produce a loose tilth to a depth of 10 inches. Cement had then been spread on the surface and thoroughly mixed in with rotary machines. Water had been added from pressure distributors, and after further mixing the whole base was compacted in a single lift to give a finished thickness of 8 inches. Progressive rolling had been adopted, employing three machines of increasing weights.

After compaction, the surface had been kept damp by repeated applications of water until the seal coat of 35 per cent bitumen emulsion had been applied a day or so later. The pavement had been completed by a two-course bituminous surfacing consisting of a 2½-inch base-course of asphalt or bitumen-macadam with a 1¼-inch hot-rolled-asphalt wearing-course.

Several types of mixers had been employed, ranging from small rotary hoes attached to Fordson tractors, to a purpose-built self-propelled Seaman's Pulvi-Mixer. Other types had included an older version of the latter machine which was towed by a separate tractor and had the rotor driven by an independent power unit. Another machine which had proved very useful was the heaviest model of rotary hoe attached to a TD.18 crawler tractor. As would be appreciated, the material resulting from the combination of the existing pavement components was extremely stoney and it had proved too heavy for the lighter machines. The mixing efficiency for the self-propelled Pulvi-Mixer had been found by the Military Engineering Experimental Establishment to be 78 per cent. The cement had been supplied both in bulk and in bags, the bulk cement being distributed by a mechanical spreader. The rates of output mentioned on p. 449 did not seem to be really significant without the daily working time being specified. At Wisley, during a 6-week period when no factors outside the contractor's control had interrupted the work, the average output had been of the order of 4,200 square yards per day. The maximum output on any one day had been 6,300 square yards. It had been estimated that the cost of the stabilization work together with the provision of the new two-coat bituminous surfacing was approximately 70 per cent of the cost of the equivalent concrete construction, whilst the time taken to complete the work was probably less than 80 per cent of that which would have been required for the replacement of the pavements in concrete on a formation that had already been prepared.

Dr A. R. Lee said that it would probably be correct to say that the use of soil-cement for the construction of roads was one of the most revolutionary advances made in recent years in the science of road-making in Great Britain. Soil-cement was a logical development to be expected in certain districts in North America, where suitable road-stone did not exist and where for many months of the year the soil was naturally in a dry and friable condition, and could easily be mixed with cement by means of suitable machines.

When engineers in Britain had first heard of the American work they had hardly looked upon the process as a serious competitor, in Britain, to broken rock or concrete ; broken rock was available almost anywhere in the country within 100 miles of a supply ; also much of the soil in Britain contained a high percentage of clay, and it rarely dried out to produce a naturally friable material. The pioneer work of one or two highway engineers, however, particularly that of Mr H. E. Brooke-Bradley, had shown that that unorthodox process could, in certain circumstances, be

used as an excellent alternative to, and as a cheaper process than, traditional flexible and concrete methods of construction. The Road Research Laboratory had been encouraged to intensify their investigations and with the aid of their engineer friends had now produced a specification for the construction of soil-cement roads for housing estates. The Authors had described how that revolutionary process was carried one stage further. The ideas put forward were the result of active co-operation between the Road Research organization and the Military Engineering Experimental Establishment of the Ministry of Supply. The co-operation had enabled the results of laboratory investigations to be tried out on a full scale at an early and critical stage with the facilities and backing of the Ministry of Supply.

The Authors were to be congratulated on showing how the possibilities of the soil-cement process could be extended to airfield conditions. The advantages put forward in its favour in comparison with orthodox methods were the savings in the amount of material to be transported, the increased speed of construction, and the reduction in costs. Those were very real advantages today and it was gratifying to know that the Air Ministry was giving it serious consideration in relation to the construction of permanent airfields.

The information given in the Paper on the results of the tests with the loading machine raised the general problem of the structural design of runways of the flexible type. Dr Lee understood from the Authors that, in the more recent field tests, a 12-inch thickness of cement-stabilized clay base with a 2-inch bituminous wearing surfacing had been laid on a soil subgrade with a C.B.R. value of 3 per cent, and that it had been quite unaffected by 15,000 passes of a 5-ton loaded axle carrying two pneumatic-tired wheels.

That was a remarkable result and indicated the seriousness with which the flexible type of soil-cement construction for permanent runways should be regarded, even on weak subgrades. On p. 453, the Authors had stated that " in the present stage of knowledge, stabilized soil bases should be constructed to the same thickness as that required with any other type of base material." Well-compacted soil-cement might perhaps be regarded as a granular base having completely ideal interlocking properties. Dr Lee suggested that the close interlocking and high internal friction that could be obtained with a well-compacted soil-cement base might far exceed anything possible with any other granular material and thus enable it to be used in thinner layers than was necessary with broken stone and other traditional flexible-base materials. Whether or not that was in fact possible could only be decided at the present stage of knowledge by further full-scale experiments. Furthermore, how far that type of flexible construction could compare in performance with concrete for permanent airfield runways could also only be decided by full-scale experiments, coupled with knowledge of the limiting degree of irregularity of the surface which was

regarded as indicating failure. The amount of permanent deformation of a flexible pavement made by a vehicle appeared to be a function of the logarithm of the number of passes of the vehicles over any particular spot. The application of the results of the field-tests carried out by the Authors to the design of airfield pavements consequently necessitated a knowledge of the distribution of aircraft across the runway. That matter had been mentioned in a Paper by Maclean and Pollitt,[1] in which results were given for the transverse distribution of aircraft landing on a runway. The maximum concentration per foot width of a 150-foot runway was only 1½ per cent of the total traffic, whereas the corresponding figure for traffic on a 30-foot road was more than 40 per cent. That might not be typical for a permanent airfield, but it did indicate that the spread of traffic was considerable, and consequently that the frequency with which traffic ran over any particular spot might be quite low. That spread of traffic might well mean that the number of repetitions of loading for design purposes for flexible airfield construction might be surprisingly low, certainly considerably lower than was required for the design of a concrete airfield pavement. On that assumption, the huge thicknesses that were considered necessary for flexible runways on weak subgrades might not in reality be necessary at all. That point would never be settled, however, until full-scale experiments were carried out with the correct loading and distribution of vehicles.

Perhaps the Authors had considered that matter and would say whether they thought it was one on which further research would be likely to lead to economies in airfield construction.

Mr W. S. Catlow said he felt that more information should have been given in the Paper on the subject of plant. Indeed, the only mention of plant seemed to be on p. 466, where it was stated that plant had been adequately dealt with in a recently published book, " Soil Mechanics for Road Engineers." In referring to that, however, he had found that there was only about half a page on mix-in-place plant. There had been great developments since the early days of soil stabilization, when agricultural harrows had been the only plant available.

Some years before the 1939–45 war, he had been responsible for the construction of a short length of road in which the base had been stabilized with emulsion, and he remembered becoming very weary with the number of passes that the harrows had to make before it was considered that the emulsion had been properly mixed with the soil. Modern plant, of course, reduced the number of passes necessary when using the mix-in-place method; indeed, he understood that single-pass machines of American make were now on the market, and that there were prototypes of similar British machines.

It was obvious that single-pass machines were costly and of great weight

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1 " Fundamental Soil Problems in Military Road and Airfield Construction." *Géotechnique*, vol. III, p. 97 (Sept. 1952).

and power, and it was a matter for consideration whether smaller and less powerful, but cheaper, machines which did the same job in a number of passes were not a better proposition. The number of passes required was, of course, dependent on the type of machine and the nature of the soil. The great advantage of the single-pass machine appeared to be that the technique was more simple and that it was not working on the fluffed-up surface caused by previous passes, so that a consistent depth of soil was treated. The main disadvantage of the single-pass machine appeared to be its Juggernaut-like bulk and weight. The main advantage of multi-pass machines seemed to be that the cement could first be mixed dry with the soil, and that might give a more even dispersion of the cement throughout the mixture.

Doubtless there were many more factors than those which he had mentioned. It would be instructive and of practical value to have those advantages and disadvantages discussed, bearing in mind the present stage of development of the two classes of machine. It was well known that up to the present only American plant had been available, and with dollars, as always, in short supply he believed that only a limited amount of plant could be imported into Great Britain. The progress of soil stabilization was therefore retarded in Britain, at least with regard to the practical aspect of the subject. It was to be hoped that British manufacturers could be persuaded to produce those machines ; it might well be said that it was almost in the national interest that they should do so, because it was essential that the country should not lag behind in that modern form of construction. The British manufacturers, however, had to be put on the right lines now and told that either single-pass or multi-pass machines were required. Possibly the real answer was that both types were necessary to deal with different conditions and requirements.

Dr H. Q. Golder observed that the Authors had provided the finest exposition that he had ever seen of the fact that, when referring to pavements, when engineers said " rigid " they meant flexible, and when they said " flexible " they meant either rigid or brittle ! He felt very strongly that they should not just copy the American nomenclature and that it was wrong to do so. He thought that the essential difference between those types of pavement was the tensile strength of one and the lack of it in the other. They should be called " tensile " and " non-tensile," or possibly the Authors after consideration could think of some better terms.

The Authors had said on p. 451 that there would be some permanent deformation of the subgrade from compaction, consolidation, or plastic flow of the soil. Dr Golder thought that the subgrade also should be designed so that those permanent deformations did not occur. If permanent deformations which were cumulative did take place, did not that mean that there would eventually be failure, and did not that mean that the pavement had not been designed adequately ?

The Authors had used considerable space on pp. 452–3 in proving that

a soil-cement was flexible or non-tensile. Dr Golder would have assumed that.

Like Mr Wilson, Dr Golder found the arguments on pp. 461–3 rather unconvincing. Possibly Mr Cooper's experience also supported that, although he had not said so in so many words. If one got water in a hard rock base, the water did not reduce its angle of internal friction. The interchange of the expressions " lubrication " and " reduction of internal friction," and saying that the latter was due to water, was a little loose. It was known that the angle of friction was not reduced by immersion in water.

Dr Golder agreed with Mr Wilson in not believing *Fig. 7*, if that was a clean sand and gravel. If it was, and the Figure was true, then he thought that the test did not properly indicate the properties of the material which were of importance in the field.

Mr Robinson had said that the military requirement was the same as the civil, but Dr Golder thought that that was perhaps not true, and that the military problem was to have a general answer which was applicable in every set of circumstances, like the Bailey bridge. In the case of stabilized soil, there was no such solution, and he thought that the Authors had done very well in developing methods which could be applied by an engineer with some specialized training.

Mr G. H. Hodgson said that, first, he wished to ask the Authors where the dividing line came between stabilized soil and lean-mix concrete, because what he wished to describe was some experimental work which had just been completed using a 24 : 1-mix concrete.

Twelve months ago, through the kindness of the County Surveyor concerned, the Ministry of Transport, and the Road Research Laboratory, he had been given one small section in which the 24 : 1 concrete had been laid and rolled during the morning; in the afternoon a 2-inch-thick bituminous base-coat had been laid and rolled, and the road had been opened to full traffic the following morning.

Three weeks ago, he had had the opportunity of laying about 2,000 square yards, the 24 : 1 concrete being used as a substitute for pitching.

A 200-yard length had been completed in a day, and during the late afternoon a 2½-inch-thick bituminous base-coat had been laid on the lean concrete, and the strip had been opened to full traffic the following morning. Because at the time all the main passes in North Wales had been blocked with snow, that 1-day-old strip had had to carry extraordinary traffic, that coast road through Caernarvon and on to Chester being the only road clear of snow.

Mr Hodgson then displayed a series of slides, the first of which showed the 24 : 1 concrete being mechanically laid; he believed that it was the first time a Barber-Greene machine had been used for that purpose. Further slides showed the 2½-inch-thick bituminous base-coat being rolled on the newly consolidated concrete, and the Barber-Greene machine at

work, placing the lean-mix concrete and being closely followed by the 10-ton roller.

Since, in places, the depth of the consolidated concrete was 14½ inches, the lean concrete had had to be laid in two layers, and in one place, three layers, one above the other. Where more than one course was necessary, it was important to consolidate only partly the lower course before placing the upper course. That ensured a good bond between the layers.

One pass of the 10-ton roller was sufficient to consolidate the lean concrete so that the Barber-Greene machine and the lorries feeding it could pass back over the partly consolidated concrete to lay the upper course.

The lean-mix concrete, when fully consolidated, had a compaction of about 1½ inch on 6-inch layers.

A 10-ton roller had been used to consolidate the concrete and, in Mr Hodgson's view, when the bituminous material was put down the same day, one should use a slightly lighter roller, say 8 tons, for rolling the bituminous material.

It was interesting to note that the 1-day-old strip of concrete with its covering of bituminous material, 14½ inches thick at its deepest part, had successfully carried the extraordinary traffic, 22-ton cement lorries running within 6 inches of the unsupported edge of the strip.

In mixing that lean-mix concrete it was very important to see that the coarse aggregate had been well wetted. That ensured the even coating of all particles and helped to prevent segregation of the materials in the finished mix. The mix used was 1 cwt cement, 10 cwt quarry sand (3/16 inch and down), and 14 cwt graded crushed rock (1½ inch to ¾ inch). The water/cement ratio had been 1.

Another slide illustrated Mr Hodgson's theory of how the material was consolidated. When the surface was rolled, the angular particles were squashed together and became interlocked. That showed the importance of using angular material. The last slide showed how the angular particles interlocked, giving mechanical stability.

The Borough Engineer of Conway had been so pleased with the system that he was constructing 500 square yards of new road on virgin soil. The lean-mix concrete was being placed to a consolidated thickness of 4 inches, on which a 2-inch-thick layer of bituminous base-coat was laid, the final surface being fine cold asphalt.

There was an entrance to a garage which had had to be kept open, and traffic, including a 10-ton tanker, had run over the freshly placed concrete, without its bituminous base-coat, but no damage had been done to the concrete. That method of using a lean-mix-concrete base and at once putting on a bituminous top and opening the road to traffic the following day had very interesting and distinct possibilities for rapid work, whilst for airfield work, where traffic was not such a nuisance, it would be even simpler to carry out.

*** Mr H. E. Brooke-Bradley** commended the Authors for including in the Paper one of the main groups of soil stabilization processes, namely, mechanical stabilization. That method was too often ignored when considering the possibilities of stabilizing soils, although deposits or supplies of suitable material were available at economical cost and cheap haulage. It was often cheaper to effect stabilization of an inferior base by mechanical means than by the use of additives, and for that reason the comments made by the Authors on that process deserved careful consideration.

Mention was made of the proportions of cement added to the soil, ranging from 5 to 15 per cent by weight. If an excess of 10 per cent of cement was used, the process became costly and sometimes did not compare favourably with other forms of construction. During the 30-odd years in which Mr Brooke-Bradley had employed soil stabilization in road construction, on no job (other than preliminary experimental work in 1917) had it been necessary to use more than 10 per cent of cement to obtain a satisfactory result.

In dealing with the stabilization of clay soils, he had found that hydrated lime usually gave better results than cement, and on a trunk road carrying 20,000 tons per day, a heavy sodium clay had been successfully stabilized by the addition of 20 per cent waste chippings, 8 per cent hydrated lime, and 2 per cent cement.

Many clays suffered from acidity, which was undesirable since that increased plasticity, swelling, and shrinking more than a neutral or slightly alkaline clay soil. In Worcestershire, where much of the clay was found in the central and northern areas, high acidity was encountered in the flat plains. For that reason, hydrated lime gave better results than cement, but the acidity could be overcome by spreading about 1 lb. of limestone or slag dust per square yard over the site to be stabilized, prior to processing. The use of waste granular particles incorporated in heavy clay soils was extremely beneficial, particularly if the moisture content was high and had to be reduced by aeration.

Mr Brooke-Bradley agreed with the Authors that bitumen could not be satisfactorily utilized with clay soil, and its use with granular material required very careful control and reasonably good weather conditions.

There was considerable scope for special plant for processing soils, and a comprehensive machine, which combined in one operation, cutting, pulverizing, blending, and mixing, would seem to be the only satisfactory type to deal with the new technique.

Mr D. J. Maclean, in reply, referred to Mr Cooper's comment that, if flexible construction was used on soft soils with a C.B.R. of 3 per cent, 4 feet of construction was required. Mr Maclean was very doubtful indeed whether anything like so much thickness was necessary, and he thought that Dr Lee had adequately answered the point later in

*** This contribution was submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

the discussion in referring to the very much lower intensity of traffic which airfields received compared with roads. As Mr Cooper was aware, Mr Maclean had been very anxious for a long time to see the validity of those full-scale trials, where a small area was intensely trafficked, checked with actual airfields; he felt that there was need for a great deal of work in that direction.

Mr Cooper had also referred to the fact that the presence of a bituminous surfacing on a granular base might increase the resistance to shear by a surcharge effect. That seemed very likely to be the case, in which event it would be expected that a bituminous surfacing would have the same effect on top of stabilized soil.

Turning to the possible stability of mechanically stable soils on forward airfields, Mr Maclean's experience was that such airfields were usually provided with a pervious surfacing, and he believed that mechanically stable soils under those conditions were likely to be very unstable. If an impervious surfacing could be provided, mechanically stable soils might well provide the stability required.

The question of what strengths should be avoided in soil-cement was a subject on which more research was required before an adequate answer could be given, but when a soil-cement of higher strength was used it seemed that its flexural strength was being increased to a point where it might behave as a weak concrete, and it would therefore be unsatisfactory for that reason.

Mr Andrews had commented on the statement in the Paper that soil-cement had been found to be 4*d*. per square yard cheaper than hardcore, and said that he had recently been in Lancashire, where that situation did not arise. The Authors had said " on an average," and if, of course, one had granular material on one's door-step one might be able to construct a granular base for a similar price, as with soil-cement. However, transport costs were continually increasing, and granular material had not to be transported for many miles before its cost began to rise steeply.

Mr Andrews had also commented on the reference to the possible flexural failure of concrete. The Authors did not suggest that concrete would always fail in flexure, but merely used that to illustrate their argument, in order to give their views on the actual structural behaviour of soil-cement in an airfield pavement structure. Mr Maclean felt that weathering tests, which were mentioned in the Paper, were best used in conjunction with actual measurements of strength, by measuring the strength of the material after it had been subjected to a weathering test which simulated the practical conditions thought likely to occur at the site. There was a very big difference between American climatic conditions and those of the United Kingdom.

Mr G. S. Wilson had given a very interesting account of the work which his firm had carried out at Wisley. He mentioned, as one or two other speakers had done, that in his view hard rocks would, if used as a

compacted granular base, give quite a good stability, particularly under a bituminous surfacing. In the Paper, the Authors had simply suggested that the stability would be lower with hard rock, particularly when rounded, than with soft rock, and, whilst it might be true that granular bases made of hard rocks were satisfactory in roads, it should be borne in mind that airfields were subjected to much higher traffic stresses, and it was not possible to be certain that hard rock would be as satisfactory in that case. He could recall one instance of a road where hard rock had not been particularly stable. *Fig. 7* referred to a clean sandy gravel.

Dr Lee had brought out the important point that airfields were much less intensely trafficked than roads. Research on that point would be very valuable in connexion with obtaining an economical design of airfield runways.

Mr Catlow had referred to plant problems. It should be pointed out that British machines were now becoming available. Mr Maclean would leave Mr Robinson to deal with the more detailed points about single-pass and multi-pass machines.

Dr Golder had referred to the question of nomenclature, and preferred " tensile " and " non-tensile " to " flexible " and " rigid." Dr Golder might be right, but in civil engineering there were many such terms which had come into common use and, since most people understood what was meant by those terms, the Authors had felt that it was better to use them rather than to adopt a new terminology. He did not think that those who were concerned with concrete would be pleased if the term " brittle " were adopted for that material.

Dr Golder had made a very interesting point on the question of cumulative deformation and whether one should design so that there was no deformation at all. Mr Maclean's own view was that in flexible-pavement design it was desirable to design for cumulative deformation, so that the actual deformation in so many years would not exceed a certain critical value. That was a means of securing economy. If engineers were to design, as Dr Golder had suggested, to prevent any deformation occurring at all, the designs would be likely to be very expensive. Precisely the same problem arose in regard to roads.

Dr Golder had suggested that the Authors had laboured the discussion of mechanically stable soils. The reason they did so was that they had found that so many failures arose from a lack of understanding of the principles involved. Those materials were frequently unstable in practice.

Mr Hodgson had made some very interesting comments on lean-mix concrete and had given a most interesting description of experiments in North Wales. The Authors would look forward to seeing the results of that work in due course. On the practical side, the only difference between lean-mix concrete and stabilized soil with granular aggregate was that, in the case of lean-mix concrete, the aggregate was actually prepared before mixing in a concrete-mixer whereas that was unnecessary with stabilized

soil and use was made of other types of mixer which would mix efficiently, even if there was some clay present in the soil. In other respects the materials were very similar.

Mr Brooke-Bradley had contributed from his extensive experience in using soil stabilization for constructing roads in Worcestershire. The successful stabilization of the heavy sodium clay with hydrated lime and cement was a brilliant achievement and an excellent illustration of the considerable potentialities of stabilization processes.

Mr P. J. M. Robinson, who also replied, said that Mr Cooper had referred to mechanical stabilization being an advantage in places where it might be necessary to bring in, say, 2,000 tons of cement by air. It was, of course, difficult to compare hypothetical cases but, in the first place, if 2,000 tons of cement were necessary for construction, probably more than 20,000 tons of mechanically stable soil would have to be imported. Secondly, if it was necessary to import the cement by air, it would also, presumably, be necessary to import any plant by air. Consideration would, therefore, have to be given to the difference in tonnage of the cement and the plant necessary to import the large quantities of mechanically stable material.

The advantages in working and construction given by the use of cement stabilization would also have to be closely considered.

It had not perhaps been made sufficiently clear in the Paper that heavy rainfall during construction would defeat mechanical stabilization completely. The disadvantages of that from a military point of view were obvious.

Mr Cooper had asked about the thicknesses to which a single-pass stabilizer could work. It was safe to say that existing single-pass stabilizers could work up to thicknesses of 10 inches. The requirement for any greater thickness would be based only on the existing illusions concerning thicknesses of pavement. Probably no greater thickness than 18 inches would ever be necessary for an airfield. except in exceptional cases such as on peat, and that thickness would be achieved by bituminous surfacing on the cement-stabilized soil.

It was not possible to define the clay fraction which a single-pass stabilizer would mix, since the effect of the clay fraction was different on different soils, but a single-pass stabilizer had been used to stabilize soils well within the cohesive range. Probably the limit of single-pass machines at present was to stabilize soils with a Casagrande classification of CL.

Mr Wilson had referred to the mention in the Paper of 200 lb. per square inch as the general military requirement and to the suggestion that, owing to the necessity for putting military airfields into commission as soon as possible, a higher 7-day strength was required for them than for permanent airfields. The figure of 200 lb. was the estimated strength required, not at 7 days, but at the time of trafficking. Only very limited experiments had been done on the subject, and the figures quoted were intended to be

a guide more than anything else; they were not definite, because the materials themselves would vary. If a set of experiments were duplicated there would still be a variation. However, a soil-cement road had been trafficked 24 hours after construction with 2½-ton traffic and it was found that it required a 7-day strength of 400 lb. per square inch.

Mr Catlow had referred to single-pass and multi-pass machines. The problem was a very involved one, and Mr Catlow was quite right in what he had said about the bulkiness of single-pass machines. They were bulky at present, but the aim was, with the machine shown in the illustration, to reduce that bulk considerably and to make the single-pass machine no more, if not considerably less, bulky than some of the multi-pass machines. Mr Robinson could think of no advantages of having more passes than one; he did not think that it had ever been demonstrated that it was advantageous to mix cement dry. In the very limited work which had been done in trying to compare mixing-in the cement with the soil dry and the soil wet it had been found that there was no difference, and from the point of view of the technique required it was much simpler, for military work, to use a single pass.

Dr Golder had referred to nomenclature. It had been difficult enough to introduce a new process without introducing a new nomenclature as well. Dr Golder appeared to have been misled by the statement that the military requirement was the same as the civil. Mr Robinson had meant that to apply simply to the slides which he had showed when introducing the Paper, as meaning that economy in cement was equally advantageous in military and civil work. Otherwise there were many differences, though they were not major ones.

Mr Hodgson's remarks had been very interesting, because the work which he had done had largely parallelled the work with which the Military Engineering Experimental Establishment and the Road Research Laboratory had been concerned. Mr Robinson thought that the division between lean-mix concrete and soil-cement depended on whether one called the main bulk of the material "aggregate" or "soil." He thought that that would be decided by saying that when one paid for it, it was aggregate, and when one got it free, it was soil. Probably Mr Hodgson would find that he could make his lean-mix concrete with soil and get just as good results.

The Authors had used the spreader which Mr Hodgson had mentioned for various types of soil, and in many ways it was successful, but it was not adapted to spread a great thickness. Mr Hodgson seemed to have got over the difficulty by spreading a thinner layer. In the Military Engineering Experimental Establishment trials, they normally spread 10 inches, and the rate of feed into the machine in their case had not been enough to cope with the rate at which it came out of it, so that it was necessary to stop from time to time to fill it up. That led to some differential compaction and lumpiness. As for opening the road to traffic after 24 hours,

unsurfaced, they themselves had had the same experience with a very great concentration of traffic. It bore witness to the good quality of the mixed material. In the Military Engineering Experimental Establishment trials a rapid-hardening cement was used, and with ordinary Portland cement no mixes would stand up to traffic after 24 hours. With such a selected aggregate as Mr Hodgson had used, therefore, it might be questioned whether any additives were required for its stabilization.

Paper No. 5899

"Post-War Coast Protection Works at Fleetwood, Lancashire, 1946–50"

by

William Melville, M.I.C.E.

(*Ordered by the Council to be published with written discussion*) †

SYNOPSIS

The Paper gives a short history of coast erosion on the west shore of the Borough from the early nineteenth century.

In 1918, the Corporation of Fleetwood was concerned at the serious risk of flooding during times of storm, and sought expert advice. The works carried out in 1927–28 are described, together with observations on their effectiveness.

Deterioration of the foreshore and the need for more substantial works in the post-war years are discussed; in 1946 this indicated a possible expenditure of £226,000. The scheme is being carried out in stages, two of which are completed at an expenditure of £85,000 and £105,000 respectively.

These works are designed to protect the existing works by additional construction on the seaward side, in the form of broad aprons paved with igneous blocks. The promenade decking has also been substantially widened.

In 1950, a further section of the works was considered necessary, but before being authorized it became evident that the extent of the work should be increased to provide similar protection over a greater sea frontage.

Eight new groynes are nearing completion.

Reference is made to experience gained in applying the terms of the Coast Protection Act, 1949.

Introduction

Coast erosion on the west shore of the Borough has been a problem for many years, and of increasing seriousness during the second half of the 19th century when the urban area of the town, established in 1838, began to extend to the west. Defence works were constructed in 1834–35 by Sir Peter Hesketh Fleetwood for the protection of his lands at Rossall, extending northwards to Rossall Point, but only a small portion of these works still remains.

The Fleetwood Estate incurred considerable expenditure from 1876 onwards in the construction of groynes with a timber hulking, with the object of stabilizing the foreshore co-extensive with lands north of Rossall

† Correspondence on this Paper should be received at the Institution by the 15th October, 1953, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

School which had been acquired from Sir Peter, and which are referred to on p. 78 of the 3rd report of the Royal Commission on Coast Erosion, etc., dated 1911.

Further erosion and the consequent lowering of the gravel ridge above high-water mark led to serious flooding of the hinterland during times of storm; when these times coincided with high water of spring tides, such flooding by sea-water had an increasingly damaging effect upon the built-up area.

The 1925 Scheme

During the war years of 1914–18, the condition of the groynes on the west shore rapidly deteriorated, and the then Urban District Council in 1920 invited the late Mr C. N. Abernethy, M.I.C.E., to report on the conditions. Arising therefrom, consultations on the problem took place between the Local Authority and the Fleetwood Estate Ltd, and on the decease of Mr Abernethy, a further report and a scheme of protection were submitted by Messrs Lewis and Lewis, MM.I.C.E., dated the 7th August, 1925. This scheme formed the main subject of the Urban District Council Act of 1927, under which powers were obtained for the Local Authority to construct a sea wall about 1,900 yards in length.

This scheme, to the design of Messrs Lewis and Lewis, was carried out in 1927–28 and consisted of a mass-concrete sea wall founded on an alluvial clay bed generally at a level of 13·50 O.D., with a coping level of 26·00 O.D., being 11·50 feet above high water at ordinary spring tides. A concrete promenade decking, 15 feet wide, was superimposed on filled material to the back of the wall. At the same time, the existing timber groynes were repaired and extended.

The works, while restraining the lateral erosion of the coastline, have not proved effective against vertical erosion, and periodic exposure of the foundations has occurred. This problem has been dealt with by underpinning the foundations and by major schemes of protective reinforced-concrete aprons with steel sheet toe-piling.

In January 1938, the Council placed the resources of the Corporation at the disposal of the Rossall School Authorities to safeguard the sea defences, co-extensive with the school property, which were disappearing rapidly during an exceptionally stormy period. A portion of the works affected was that constructed in 1834–35, already referred to.

When the works were temporarily secured, the Local Authority and the Rossall School Corporation entered into negotiations, arising from which the Fleetwood Corporation accepted the responsibility of making good the damage, and of the future maintenance of all the works. The Local Authority thus became responsible for the protection of the whole of the sea coastline—about 4½ miles in length—within the Borough.

The 1946 Scheme

During the period 1939–45, further deterioration of the foreshore continued, and maintenance work was hampered by works of military defence, which restricted access to the foreshore, and by shortage of labour and materials. The exposure of the Rossall revetment to heavy storms during this period and the permanent lowering of the foreshore resulted in a weakening of this old defence work, and in February 1946, a major collapse occurred. Emergency work was undertaken, to limit the extent of the damage, at a cost of about £6,700.

It was then obvious that the whole system of sea defence on the west shore of the Borough should be reviewed, since works of an emergency nature were arising with regularity ; after informal consultations with the County Surveyor, a comprehensive report was presented to the Council on June 24th, 1946. The extent of the proposals was such that the Council asked for the report to be considered by a Consultant Engineer. Sir Cyril Kirkpatrick, M.I.C.E., and Mr Jack Duvivier, M.I.C.E., were jointly commissioned to examine the proposals, reporting thereon in November 1946.

The Consultants' report approved the proposals of the Borough Engineer, who then prepared a scheme, at the then estimated cost of £226,000, for improvements to the sea defence works to be carried out in stages over a period of time.

Post-war coast protection works in Fleetwood have been designed with two objects. First, the safeguarding of the existing protective works in their present position by additional construction on the seaward side, and secondly, to broaden such defences to reduce the quantity of spray-water which overtops the works during storms.

The first is an extension of the policy which has been progressively forced on the Council for some years, arising from practical experience. Low-water mark is about 500 yards from H.W.M.O.S.T. and the tidal range is from −15·00 to +17·00 O.D. This large expanse of foreshore is intermittently covered with sand and gravel but the underbed consists mainly of alluvial clays. These clays, at times denuded of beach material, are subject to rapid erosion with little advance warning. As a result, the beach level near the defences has progressively lowered, and the only practical solution has been the construction of sloping aprons to safeguard the existing wall foundations and to reduce the depth of water against the wall. In other words the existing wall has become incorporated as a substantial coping in the design of the newer construction.

From observations on other parts of the Fylde coast it would appear that there is a point at which vertical erosion is arrested and the beach appears to become stable at an approximate level of 5·00 O.D. A new series of eight long groynes to maintain a sand beach above this level is included in the post-war works, and the reconditioned existing groynes

are intended to stabilize the upper beach. The widening of the decking at the rear of the wall was made imperative by physical conditions—the general level of the land in the borough is about 17·00 O.D., whilst the sea wall is 26·00 O.D.

Any large quantities of sea-water crossing the defences cannot return to the sea other than by gravity at low tide partly through the sewerage system or by the provision of special outlet drains.

The original 15 feet of promenade decking is now 40 feet wide and this is found to be little enough for its purpose. Since the completion of a length of widened promenade there has been a considerable reduction in the amount of spray-water to be dealt with on the landward side.

The 1946 scheme of £226,000 provided for concrete aprons, steel sheet-piles at the toe, widening to promenade decking and new groynes; the first instalment, at an actual cost of £85,000, was put in hand in August 1947. These works were completed in 1949 after aggravating delays caused by cement and labour shortages. The second instalment, at a cost of £105,000, followed in 1948, and is now completed except for a portion of the groyne work.

The 1950 Scheme

A third instalment, estimated to cost £95,500, was contemplated in 1950; but delays occurred in conforming to the procedure required by the Coast Protection Act, 1949, and before arrangements were completed to put the work in hand, further deterioration occurred to other portions of the foreshore giving rise to an emergency following serious storms in November/December 1951. In making the damage temporarily secure it became evident that the 1950 scheme of works required substantial extension both at Rossall Beach and north of Rossall School. These are estimated to add £150,000 to the 1950 costs bringing the total actual and estimated cost of post-war works to £435,500.

Fig. 1 is a diagrammatic plan showing the site of the most important coast protective works in the Borough, the dates of the recently constructed works, and the works which remain to be put in hand.

Fig. 2 (*a*) indicates the nature of the erosion of the alluvial clays of the upper foreshore when denuded of beach material during storms. In this illustration the erosion is advancing towards the wall foundation. The gravels in the foreground are signs of a return of beach material which protects the remaining clays from further erosion. The materials near the wall in the background have been carted and tipped on the foreshore to augment the supply of beach material as an emergency measure.

Fig. 2 (*b*) illustrates a further stage in the erosion of the alluvial clays. The steel sheet-piling at the toe of the sloping apron is reaching a stage when it becomes unstable.

Fig. 3 (*a*) shows the first completed post-war works. The concrete apron

Fig. 2 (a)

Fig. 2 (b)

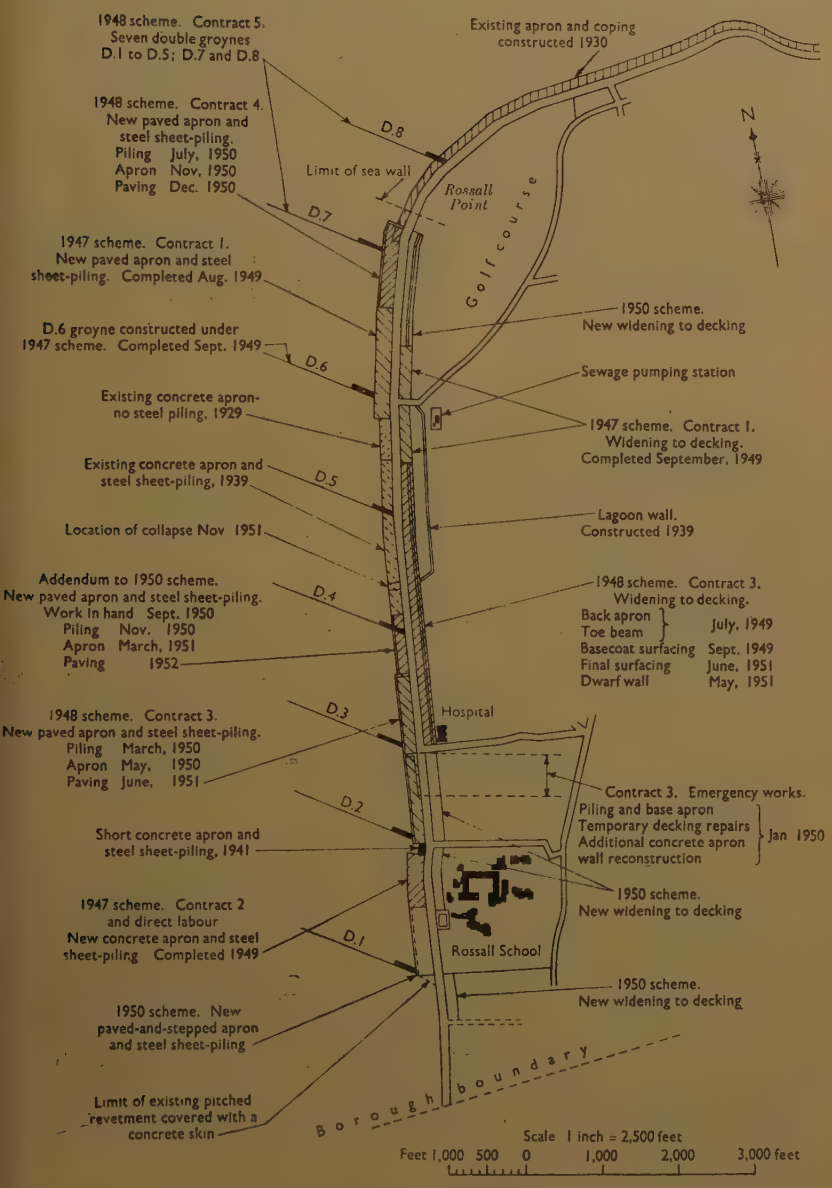
TWO STAGES IN THE EROSION OF ALLUVIAL CLAYS OF UPPER FORESHORE

Fig. 3 (a)

Fig. 3 (b)

TWO VIEWS OF COMPLETED POST-WAR WORKS

Fig. 1



THE MAJOR PROTECTION WORKS AT FLEETWOOD

has a long upper slope and a steeper lower slope which is protected with steel sheet-piling at the toe at a level of 5·00 O.D. The apron is paved with igneous stone blocks grouted with cement-mortar. The apron is divided into sections by steel sheet-piling to minimize the extent of any possible damage in the future. The beach material is returning to cover the lower portion of the apron.

The first groyne is new and incomplete. The planking is being covered and made ready for the next lift.

Fig. 3 (*b*) gives a general view of the widened promenade. Owing to cement shortages it was decided to substitute bituminous macadam, 4 inches thick, for in-situ concrete paving. The macadam is finished with a fine asphaltic surface and is to be grouted periodically to prevent penetration of sea-water.

Coast Protection Act, 1949

The following notes on the financing of the works and experience gained in applying the Act are submitted.

On assurance from H.M. Government that expenditure on approved capital works after 15th August, 1945 (" V.J. Day "), would rank for grant, the 1947 and 1949 portions of the scheme were put in hand. The Ministry of Health (now Ministry of Housing and Local Government) recommended grant-aid based on the local rate levied in respect of sea defence works, and the Borough has so far qualified for a substantial percentage grant. Under the 1949 Act, however, the Local Authority has to seek contributions from owners of land benefiting by the scheme and to decide whether to proceed by means of a " works scheme " or otherwise. If the Local Authority fails to secure contributions which the Ministry judge should have been forthcoming, then the amount of the grant may be adjusted accordingly.

Every coast protection scheme requires individual consideration. In many cases it is relatively easy to determine the extent of the lands protected and to assess the value of such protection. In other cases there is a large number of individual small-plot owners to be satisfied as to the extent of their contributions. The topography of the Borough of Fleetwood is such that the whole town is dependent upon the security of the sea defences, and it has been suggested that all should contribute in proportion to rateable value through the general rate. The actual site of the works is on land in the ownership of three parties only, and these are willing to give the land and grant facilities as a contribution to the cost of the works.

The full severity of the storm on the 30th and 31st January, 1953, which had such serious consequences on the east coast was not felt on the Lancashire coast.

The Tables predicted a tide of 27 feet 11 inches at 12.16 p.m., on the 31st January at Fleetwood, and the recording gauge registered 29 feet 4 inches.

The wind direction over the flood period was due north averaging 55 miles per hour with gusts of 75 miles per hour.

It will be noted that the tide was only 17 inches above its predicted height and although the sea was very rough with high waves, the coast protection works did not suffer any damage.

The Paper is accompanied by four photographs and one diagram, from which the half-tone-tone page plates and the Figure in the text have been prepared.

Paper No. 5901

"Coast Protection : Some Recent Works on the East Coast, 1942–52"

by

Jack Duvivier, B.Sc.(Eng.), M.I.C.E.

(*Ordered by the Council to be published with written discussion*)†

SYNOPSIS

On the Lincolnshire Coast a special significance attaches to the problem of coast protection as a result of recent disastrous floods. The land is low-lying, in some parts below the level of high water, and the foreshore consists of a wide expanse of clay and warp overlain by a shifting bed of fine sand. The natural defence against erosion and inundation consists of a row of sand-hills, maintained for generations by a simple process of thorning, and supplemented in places by a secondary clay bank.

Before 1943, the artificial defences comprised stepped walls of mass concrete. The groynes were neglected and ineffective. A substantial scheme of new construction and reinforcement of the old defences was carried out between 1943 and 1949.

At Sheringham the foreshore consists of chalk overlain by a bed of grey flint pebbles down to half-tide level and sand beyond. The sand cliffs and the high land on which the town is built are protected by massive concrete walls and groynes. The walls were undermined by a gale which swept the beach clear of its protective covering of shingle.

At Cromer the chalk is at a lower level and the walls are founded on beds of sandy clay, sand, and gravel. The beach is a mixture of sand, flints, and pebbles eroded from gravel pockets in the cliffs. In other respects conditions are somewhat similar to Sheringham. The wall was undermined and very nearly destroyed by a severe gale.

Southwold is surrounded by marshes on three sides. The old defences comprising timber breastworks and groynes were in a very worn and weak condition in 1946 and were extensively damaged and further weakened by a severe storm in 1947.

Aldeburgh is bounded by marshes and the River Alde on its south side and much of the town is at a low level and very close to the sea. Continued encroachment by the sea necessitated artificial defences for the protection of the town whilst a sheet-piled wall built at the south end of the town in 1937 without groynes was rapidly becoming unsafe.

The contrasting conditions at these five places and the problems with which the engineers were faced in protecting them against further encroachment by the sea are described and the Paper contains a description of the different methods that were adopted to achieve this purpose. Costs are given in each case.

Introduction

This Paper is an account of some recent problems which have confronted coast protection authorities in three of the eastern counties of England since the end of the 1939-45 war, in the fulfilment of their statutory

† Correspondence on this Paper should be received at the Institution by the 15th October, 1953, and will be published in Part II of the Proceedings. Contributions should be limited to about 1,200 words.—Sec. I.C.E.

obligations to prevent erosion or inundation of coast lands within their areas, by the sea.

Reference is made to the physical and geological conditions of the coast in five selected areas between the Humber and the Thames so far as they have a bearing upon the problem of coast protection, and the Author has endeavoured to show that the design of a sea wall or a series of groynes is not merely a haphazard selection of one of a large number of alternative designs, all equally feasible, but that it follows from the logical application of certain established principles to the special conditions which are encountered at the site.

Special reference has been made to groynes which constitute such an important part of most schemes of coast defence, and the Paper describes the different behaviour of groynes under various conditions of foreshore and exposure.

Lastly, costs have been given for the various types of sea wall, breastwork, toe walls, aprons, and groynes constructed at the various sites.

The question of cost is uppermost in the minds of local authorities where sea defence works are concerned, and although substantial grants are now being made under the Coast Protection Act by the Government and by the County Councils and, to a limited extent, by the frontagers themselves, the balance of cost falls on the ratepayers in the borough or urban or rural district concerned.

The costs given in this Paper have been abstracted from the final measurements of the various contracts, with the addition of appropriate percentages in each case to cover such items as access, insurance, and an office for the contractors and resident engineer. They do not include engineers' fees or supervision.

The Lincolnshire Coast.

Mablethorpe to Skegness

Fig. 1 is an aerial view of a typical stretch of Lincolnshire coast. The land is mainly agricultural with occasional small towns and villages sheltering behind the sea embankment. The land is low-lying for several miles back from the coast, and widespread inundation has occurred from time to time when the natural or artificial defences have been breached by the sea.

On this coast the Mean High-Water Spring-tide level is approximately 10·50 feet above O.D. (Liverpool Bay), and since part of the land lies between 8·00 and 11·00 feet O.D., while some areas near the sea are as low as 7·00 O.D., coast protection is clearly a factor of considerable importance in the lives of the inhabitants.

The Land Drainage Act of 1930 authorized drainage and catchment boards to carry out such essential works in the sea or estuary as might be

necessary to secure an adequate outfall for the main river. The Minister of Agriculture and Fisheries was empowered to make grants towards the expenditure incurred, amounting in exceptional cases to as much as 80 per cent of the cost of the works.

The responsible authority under the Act for the length of coast under consideration was the Alford Drainage Board—a small board with a great weight of responsibility towards the local farmers and residents.

The " main river " comprises a number of drains or dikes, with tidal outfalls through which the land-water is discharged into the sea. The maintenance of the drains and of the sea outfalls entails, in effect, the maintenance of the coastline as a whole.

Parts of the coast are bordered by sand-hills up to 40 feet in height. From earliest times they have been maintained by a primitive but effective process known as " thorning " ; bundles of thorn brushwood or " kids " cut from the hedges are dug into the seaward slopes of the hills to gather and hold windswept sand driven in from the foreshore. The Romans, who were not without experience in such matters, provided, as a second line of defence, a clay bank situated some distance back from the coast. The " Roman Bank " still exists, partly as a separate embankment and partly as a core to the sand-hills, and is an important feature of the sea defences.

The foreshore is flat, the distance from the sand-hills to low-water mark being about 1,000 feet. It consists of post-glacial clay and " warp " covered with a varying thickness of medium to fine sand. Borings made in 1946 at the site of a new sea wall revealed, at one place, a layer of warp only 5 feet thick, overlying fine sand and silt more than 50 feet deep.

There is no pronounced southward littoral drift along the North Lincolnshire coast such as is found along the Holderness coast of Yorkshire. The sand is constantly shifting and is not usually present in sufficient quantity to protect the underlying strata without the assistance of artificial works ; during on-shore gales the sand is usually scoured away from the upper part of the foreshore, leaving hollows which are gradually filled again with sand after the gale has subsided.

The reason for the relative absence of littoral drift is that beach material, whether shingle or sand, is mainly derived from the erosion of cliffs, and there are no cliffs between the Humber and The Wash to replenish the supply, although shingle beaches exist here and there, derived from the erosion of local beds of boulder clay.

Some sand may be brought in during periods of easterly or north-easterly winds from outlying sandbanks such as the Haile Sand or the Knoal, and carried southward by the flood tide or blown southward by northerly winds ; some sand is also derived from the erosion of the sand-hills, but the construction during the past 50 years of long lengths of sea wall have interrupted this source of supply.

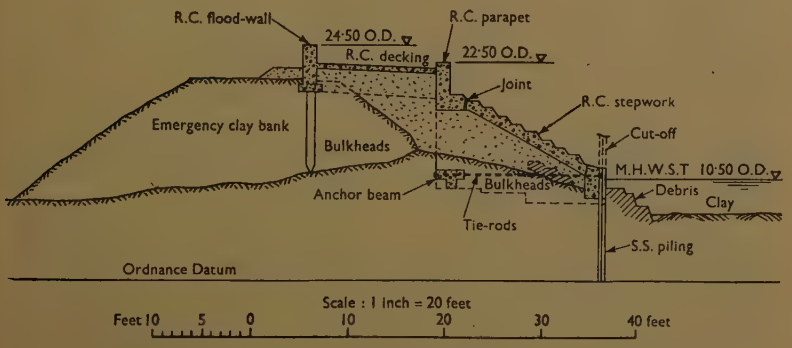
The clay and warp are naturally eroded whenever they are exposed to wave action. At Trusthorpe the clay stratum has been eroded to a depth

of 4 feet in 20 years, whilst at Huttoft the warp has been eroded to a depth of 4 feet in less than a year over a wide area.

The sand-hills have permanently disappeared from many parts of the coast and have been replaced by clay embankments, timber hulkings, and, during the pre-war years, by mass-concrete stepwork.

Before 1943, the maintenance of 19 miles of coastline had been carried out departmentally by the Drainage Board, who employed two gangs of seven or eight men under the direction of two foremen highly skilled in the arts of thorning and faggoting. In 1942, the Board sought the Author's advice in connexion with a short length of stepwork which had been breached by a storm. This was sealed-off without difficulty, but shortly afterwards the stepwork defence of Trusthorpe bank was almost

Fig. 2



Trusthorpe Bank, Lincolnshire. Reconstruction of Breach

completely destroyed by a severe storm for a distance of 600 feet, and considerable damage and flooding occurred. A borrow-pit was opened at the back and the breach was closed with clay while a scheme for reconstructing the breached wall was being prepared and materials were being ordered. Steel sheet-piling, 15 feet long, was then driven across the breach to a level of 4 feet 6 inches above M.H.W.S., behind which a stepped reinforced-concrete wall (*Fig. 2*) with bulkheads, decking, and flood wall was constructed. The sheet-piling was burnt-off on completion to the level of the reconstructed stepwork.

Sliding joints were provided to allow the stepwork to settle in panels independently of the bulkheads and parapet, the former being founded on undisturbed clay, the latter spanning as a beam between bulkheads. Settlements of up to 1 inch occurred in 5 years and the gaps were sealed-off from time to time with a mastic compound. The wall, decking, and filling cost £26 12*s.* 0*d.* per foot run.

In view of the critical condition of the sea walls in the Trusthorpe area

and the obvious need to reinforce substantially the defences at other threatened parts of the coast, the Board instructed the Consulting Engineers to inspect and report on the state of the whole of the coastline from Mablethorpe to Skegness. The report recommended that an extensive scheme of new construction, rehabilitation of existing defences, and groyning work should be carried out under a 5-year plan. The proposals were approved and the works were carried out under eight separate contracts; they were completed in 1949 and the total cost was £338,000.

It is significant that, before 1944, no attempt had been made to reinforce the defences of the northern part of the area by means of a planned and co-ordinated system of groynes, although it should have been evident from the results obtained from a series of groynes which had been erected at Ingoldmells some years previously by the late Mr R. F. Grantham, M.I.C.E., that the flat sandy foreshore of the Lincolnshire coast can be persuaded to react favourably to the provision of long low groynes laid-off to a certain alignment.

The groynes which existed in 1943 in the Trusthorpe and Sutton-on-Sea area were partly derelict, situated much to far apart to be of any real value, and pointed in the wrong direction.

Under the 5-year plan, 1¼ mile of new stepped reinforced-concrete sea wall was built between Huttoft and Mablethorpe, ⅛ mile of sheet-piled wall at Chapel, and 2½ miles of existing wall were adapted to suit the permanently lower level of the clay; protection against damage or destruction by undermining was obtained by the construction of toe walls, carried down to a depth of 4 feet into undisturbed ground, and reinforced-concrete aprons built to a slope of 1 in 2½. This appears to be about the steepest slope at which good concrete can be obtained in maritime works without having recourse to top shuttering or to a stepped form of construction at extra expense; a steeper slope requires an excessively dry mix which is liable to be porous and is subject to sulphate attack on exposure to seawater.

In cases where the old mass-concrete stepwork was worn, cracked, or hollow it was grouted up, cavities were filled with concrete and the whole was provided with a new skin of concrete 9 inches thick, reinforced with two layers of mesh and dowelled to the underlying work. Expansion joints were introduced into the encasement at regular intervals, coinciding wherever possible with cracks in the existing work. At intermediate expansion joints a plane of weakness was formed in the underlying concrete by cutting a deep chase through the old stepwork perpendicular to the wall; by this method, unsightly irregular contraction cracks, which are a source of weakness in a sea wall, were avoided.

A remarkable feature of these old mass-concrete stepped defences was the enormous cavities which could develop under the stepwork before they were detected or before a length of wall collapsed.

Some of the difficulties encountered in the construction of the new sea

Fig. 1

THE LINCOLNSHIRE COAST—HUTTOFT TO SUTTON-ON-SEA

Fig. 5

Sheringham, Norfolk

Fig. 6

The Cliffs between Overstrand and Cromer, Norfolk

Fig. 8

Cromer, Norfolk

Fig. 10

SOUTHWOLD, SUFFOLK

Fig. 12

ALDEBURGH, SUFFOLK

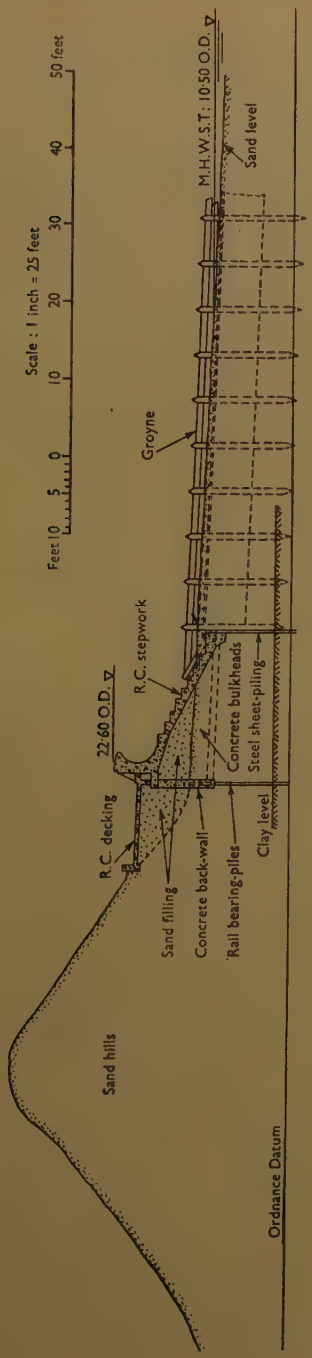


Fig. 3

SUTTON-ON-SEA, LINCOLNSHIRE. NEW SEA WALL AT CHURCH LANE

wall at Huttoft necessitated by the erosion of the warp and the formation of deep lagoons, and the methods which were used to deal with these threatening and unpleasant features, were described in the Author's Paper on " The Problem of Coast Erosion." [1]

The design adopted for 1,000 feet of new sea wall built at Church Lane, Sutton, under Contract No. 7 at a cost of £22 10*s.* 0*d.* per foot run, is shown in *Fig. 3.* The earlier walls were built to slopes ranging from 1 in 1½ to 1 in 5. Observation of the behaviour of these earlier designs under varying conditions of storm and tide showed that there was little, if anything, to be gained by adopting a profile flatter than 1 in 2.

The Sutton wall is self-anchoring and stable in the event of scour, and the design was evolved partly with the object of protecting the sand-hills at the earliest possible stage during construction. The sheet-piling was driven first, followed by the construction of the bulkheads and back wall, and the depositing and consolidation of sand backfilling. This completed the first stage of the work and removed the danger of a breach in the sand-hills. The compartments were then filled with sand one at a time and sealed with a layer of concrete ground-centering, 3 inches in thickness. The steps were cast in lengths of 25 feet and reinforcement was left projecting from the top step to bond with the superstructure, which was cast separately as the final operation. The promenade decking was added later at the expense of the local council.

The only instance in which it was necessary to depart from the stepped self-supporting concrete walls which had been adopted throughout the northern part of the area, was in connexion with the protection of Chapel Point, a salient projecting an appreciable distance seaward of the coast-line to north and south, for which the firm were asked to submit proposals for an emergency scheme following upon the collapse of the old stepped-faggot defence.

The instructions were issued later in the year, and there would clearly have been insufficient time to achieve the necessary measure of protection by means of a concrete wall before the advent of winter gales.

In 1947, steel was plentiful and could be obtained without delay. A design was therefore submitted comprising a simple anchored wall of steel sheet-piling, reinforced at the Point by a secondary low-level sheet-pile apron together with a waterproof decking and flood wall. (*Fig. 4.*)

The work was carried out under Contract No. 9 at a cost of £31 10*s.* 0*d.* per foot run. The only trouble experienced during the construction of this wall occurred when the high-level sheet-piling had been driven round a part of the perimeter. A rough sea set up vibration in the piling, and a film of water percolated between the piling and the clay, with the result that several of the piles settled several feet into the underlying clay under their own weight and had to be withdrawn and held until the steel waling

[1] Maritime Paper No. 8, Instn Civ. Engrs, 1947.

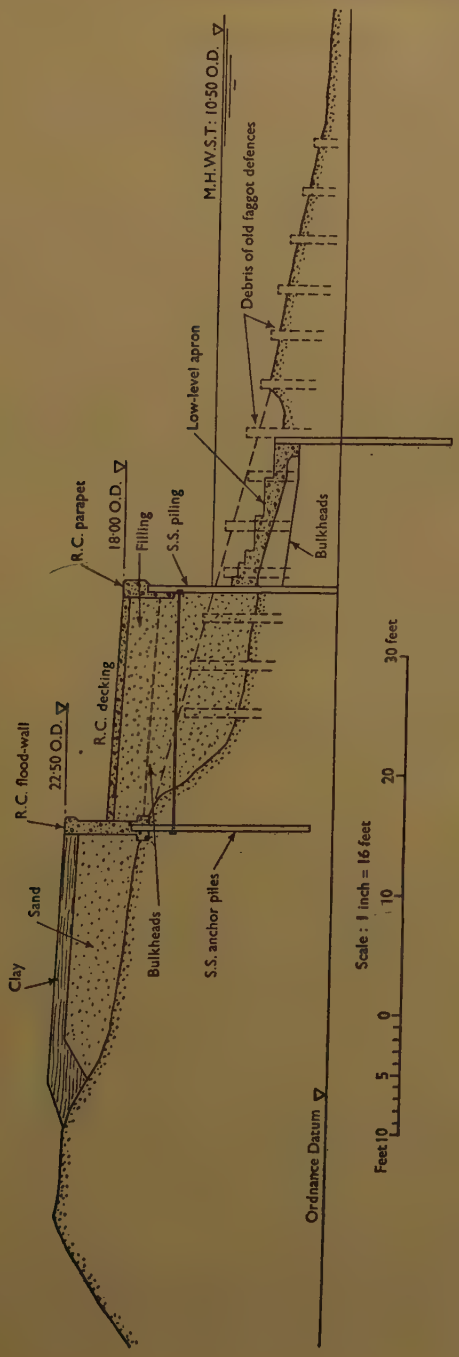


Fig. 4

CHAPEL ST LEONARDS, LINCOLNSHIRE. NEW SEA WALL AT CHAPEL POINT

had been attached. Clay for filling behind the piles was obtained from a nearby borrow-pit.

Forty-four new groynes were constructed under the 5-year plan at Trusthorpe, Sutton, Huttoft, and Ingoldmells at a cost of about £5 to £6 per foot run.

The timber used was incised-and-creosoted Douglas fir. The groynes consisted of main piles at 6-foot to 7-foot-6-inch centres with an 8-inch-by-6-inch waling about 12 inches above the beach, the timber sheet-piles being spiked to the waling, the planks being spiked or screwed to the main piles above the level of the waling. It was found by experiment that the alignment best suited to local conditions was 20 degrees south of a line perpendicular to the foreshore. The average length of the groynes was 300 feet and their average distance apart was also 300 feet.

Pile-driving in the clay was comparatively easy, but it was found necessary to have recourse to jetting when driving through sand and silt, in order to obtain the required penetration with the sheet-piles which were of 9-inch-by-3-inch and 9-inch-by-4-inch timbers, and were, in exceptional cases, up to 11 feet long.

A satisfactory expanse of firm sand has been gathered and held by these groynes, as can be seen in *Fig. 1*.

All works carried out under the 5-year-plan were grant-aided by the Ministry of Agriculture and Fisheries to the extent of 75 per cent of the cost.

The Contractors for the work were Messrs W. G. C. Hammond, Ltd, of Mablethorpe, and Messrs William Moss & Sons of London.

The Norfolk Coast

The coast from Weybourne eastward towards Mundesley consists of glacial material overlying chalk. The cliffs range in height from about 40 feet at Weybourne to as much as 250 feet east of Cromer (*Fig. 6*).

The chalk dips towards the east; at Weybourne it forms the lower part of the cliff to a height of about 12 feet above the beach; at Sheringham it underlies the shingle beach at a shallow depth and outcrops in the lower foreshore, whilst at Cromer it lies at a depth of about 7 feet below mean sea-level.

The cliffs are composed of sand, clay, and other soft material which is rapidly eroded whenever the sea has access to it. Records extending over 100 years or more show the average rate of erosion of unprotected cliff between Weybourne and West Runton to have ranged between 2 and 4 feet per annum.

At Sheringham the beach is composed of large grey flint pebbles which extend from the cliffs at a steep slope to about half-tide level; beyond the pebbles the lower foreshore extends at a flat slope to low water, and consists of chalk and flints with a covering of sand of width and depth varying

with the season and the locality. In a rough sea the sound of the flint pebbles being rolled up and down the beach can be heard a long way away, and a considerable amount of abrasion of concrete walls and aprons and timber slipways is caused by this movement.

East of Sheringham there is less shingle and what there is is usually found mixed with coarse sand, forming an evenly graded beach to about half-tide level, with chalk and fine sand beyond. The proportion of sand to shingle increases towards the east and the beach develops the different characteristics of a gravel beach.

The range of spring tides is 14 feet and the prevailing littoral drift is from west to east, but this is sometimes reversed for periods of several weeks, as one would expect it to be, in view of the exposure of the coast to winds from both easterly and westerly directions.

Cromer and Sheringham have protected their frontages by sea walls and groynes, and works on a more modest scale exist at Overstrand and Mundesley, but the rest of the coast is unprotected and the erosion of the cliffs has continued unhindered by the works of man.

Sheringham

The greater part of the present defences are between 50 and 60 years old and consist of mass-concrete walls, rising, at the centre of the town, to a height of more than 50 feet above the beach; they are founded on chalk.

The groynes are mostly constructed in concrete, but some were built of timber in whole or in part. Their lengths range between 400 and 250 feet and their distances apart ranged from 250 feet to as much as 900 feet. Within the limitations imposed by their irregular spacing, they have been very successful in holding both shingle and sand (see *Fig. 5*) but suffered from lack of maintenance during the recent war.

The excessively wide spacing of 900 feet between the two groynes at the centre of the town was due to a repeated failure to reconcile the interests of the local fishermen, who naturally dislike any sort of obstruction on the foreshore, with the requirements of coast defence.

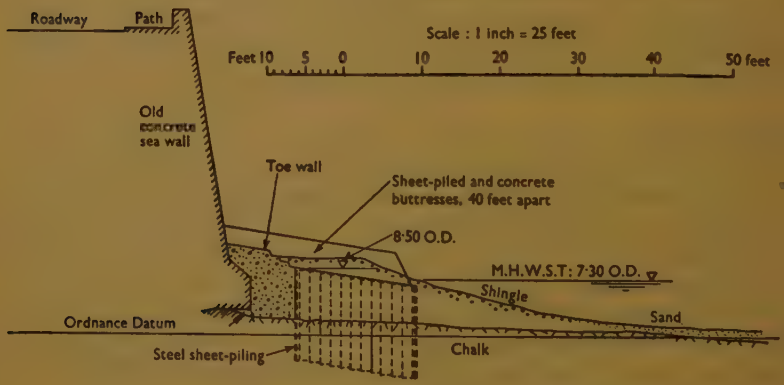
During the severe winter of 1946-47, prolonged easterly gales flattened the beach and drove much of the shingle away towards the east. The chalk was exposed over a considerable area and scoured in places to a depth of 3 feet, severe frosts assisting the process of erosion by causing disintegration of the surface, with the result that long lengths of sea wall were undermined. Fortunately, there can have been very little active earth pressure behind, since the walls neither overturned nor settled, and the situation was saved by a reversal of drift which occurred when the weather moderated. The condition of the defences naturally caused considerable alarm as some of the worst undermining occurred where the wall was highest, and a failure would have caused great damage to the town. The Author was consulted in June 1947, by which time the beach had returned more or less to its normal condition and the cavities under the wall were

buried under a pile of shingle as much as 15 feet in depth. Fortunately, the local authority had had the foresight to obtain a complete photographic record of the wall when conditions were at their worst, and the photographs were extremely useful when ways and means of repairing the damage were under consideration. The scheme of repair eventually decided upon is shown in *Fig. 7*.

It is always troublesome, and often a risk, to open up a deep excavation in front of an existing defence, but it was considered that the shingle which had presumably been driven into the cavities under the wall, would not be sufficiently consolidated to serve as a suitable foundation.

A scheme was therefore prepared which consisted of driving a row of

Fig. 7



SHERINGHAM, NORFOLK.
SECTION OF UNDERPINNING AND ELEVATION OF BUTTRESSES

steel sheet-piles, 12 feet in length, parallel to and 6 feet away from the wall, the excavation of the shingle between the piling and the wall down to the level of the chalk, the removal of all loose shingle from the cavities, and the concreting of the void so as to form a permanent sheet-piled toe 8 feet deep.

In addition, to compensate for the depth of chalk that had been scoured away and, at the same time, encourage the distribution of beach material as evenly as possible against the wall, a series of counterforts or buttresses were planned, 40 feet apart and 15 feet in length, consisting of cofferdams of steel sheet-piling, 8 feet in width, driven into the chalk and filled with mass-concrete after the removal of the shingle core.

Work on the toe wall and buttresses was commenced in March 1948, and completed in July 1949. The toe-wall hearting was removed by drag-scoop, the chalk cleaned up, and the toe wall concreted in short lengths. Some of the excavation had to be taken out two or three times and a few

partly driven sheet-piles were bent by rough seas and had to be replaced. Apart from this the work was completed without undue difficulty or damage. All the plant operated from the roadway above. Heavy-weighted covers of timber and steel were used to prevent the buttress cofferdams from being backfilled by the sea during excavation which could not be completed during a single tide. The local prejudice against groynes was overcome to some extent and three new steel-and-timber groynes were constructed to equalize the spacing between existing groynes.

The groynes consist of interlocking steel sheet-piling driven 2 to 3 feet into the chalk, every eighth pile (at 10-foot-6-inch centres) being a box pile driven deeply into the chalk to provide the necessary anchorage and support against overturning. Planking above the sheet piling consisted of 4-inch timbers of incised-and-creosoted Douglas fir bolted to the box piles.

The groynes were aligned perpendicular to the sea wall and have gathered and retained a satisfactory accumulation of shingle and sand.

The cost of underpinning the main sea wall, including the construction of the toe wall and buttresses shown in *Fig. 7*, amounted to £24 per foot run. The steel-sheet-piled-and-timber groynes cost £6 per foot run.

A considerable amount of other work was done to the sea defences, including the reconditioning and partial reconstruction of all the existing groynes, and the cost of the entire scheme, which was not finally completed until early in 1951, amounted to £47,000.

The Contractors for the work were Messrs May, Gurney, & Co. Ltd, of Norwich.

Cromer

The existing defences consist of a heavy gravity-wall, with semi-circular bastions at intervals, and six timber groynes 275 feet to 640 feet in length, 600 feet to 2,000 feet apart. (*Fig. 8.*)

The oldest part of the wall, just west of the pier, dates back to 1845 and was constructed of flints in lime-mortar. The remainder of the wall was built about 1900 to the design of Messrs Douglas and Arnott and consists of mass concrete. The cliffs were then dressed back to stable slopes, and seeded with grass and shrubs. The town lies at the back and several large hotels extend to the very edge of the cliff. None of the sea walls are founded in the chalk.

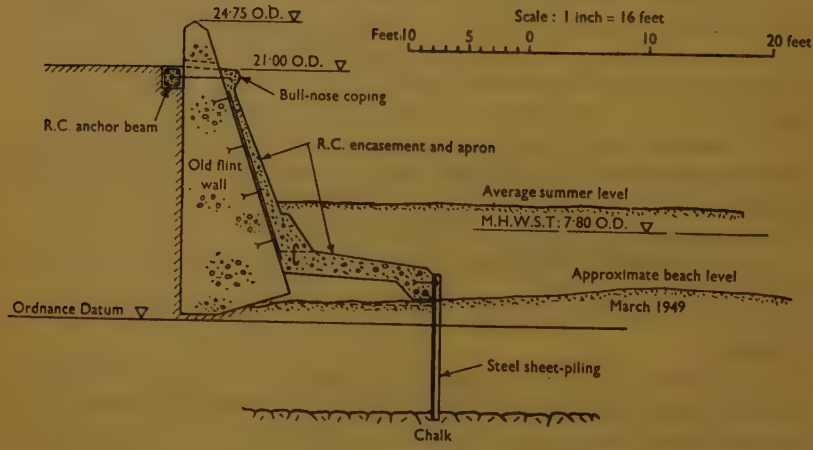
The beach consists of sand and shingle derived from the erosion of the cliffs between Sheringham and Cromer, with sand predominating—the shingle displays little, if any, tendency to separate from the sand as it does at Sheringham. It has the characteristics of a sandy beach and is flattened and the material drawn seaward by on-shore gales, exposing the underlying strata—in this case clay and flints. There appears to be an optimum percentage beyond which shingle is separated from sand by a rough sea.

The Cromer defences escaped damage in the winter storms of 1946-47

which undermined the Sheringham wall, but suffered badly during a very severe northerly on-shore gale on the 1st March, 1949. This gale will long be remembered for its destructive effect upon the defences of the east coast, owing to the abnormal level of the tide at high water which, at Grimsby, was recorded as having reached 6 feet above predicted high-water level. The foundation of the wall was exposed for a length of 1,400 feet and the wall was undermined to a depth of 2 feet and in places was left without support over half its width. The fabric of the old flint wall was also damaged and a subsequent investigation showed the lime-mortar in the hearting of the wall to be badly deteriorated. It also resulted in 54 feet at the in-shore end of one of the groynes being carried away.

A cross-section of the flint wall is shown in *Fig. 9*. It will be seen

Fig. 9



CROMER, NORFOLK.
REINFORCED-CONCRETE APRON AND ENCASEMENT OF OLD WALL

that the back is vertical and the face has a batter of 1 in 3, the centre of gravity lying well back from the toe. Considered as a wall, and not as a retaining wall, it would be just stable if undermined to a depth of 4 feet 6 inches—50 per cent of the base width—and any active earth pressure at the back would have caused it to overturn. Evidently the back-filling had consolidated, in the course of the 100 years or so which had elapsed since the deposit, to such an extent as to be virtually self-supporting.

Had the wall failed and the sea gained access to the back-filling, a slide would almost certainly have developed which might well have affected the entire cliff face as well as the buildings at the top. Slip conditions exist in these cliffs as will be seen from *Fig. 6*, which shows the cliffs just east of

the Cromer boundary groyne. A serious slip occurred quite recently at the zig-zag cliff-path on the left-hand side of *Fig. 8*, which has necessitated the provision of deep drainage in conjunction with a piled-and-anchored breastwork.

The situation in March 1949, when the Author's firm was consulted by the Council, was evidently very worrying and fraught with possibilities of disaster.

Fortunately for the town, the weather moderated before any serious damage was done to the main defences, and a certain amount of sand and shingle was replaced and compacted by the sea in the cavities under the toe of the wall. A deep swill or gully had been scoured near the wall and this began to fill with gravel from the seaward side but at a comparatively slow rate.

The problem differed from Sheringham to the extent that the Cromer wall was founded partly on sandy clay overlying chalk at a depth of 8 feet below the base of the wall. There is no real stability in this material when it is exposed to the sea and there could be no question of constructing a toe wall down to the chalk, nor was it necessary to do so.

The scheme of repair which was approved and executed—see *Fig. 9*—comprised a reinforced-concrete apron 12 feet wide, terminating in a continuous row of steel sheet-piling driven into the chalk. The filling under the apron was sub-divided by means of bulkheads and provision was made for consolidating the sand and shingle under the toe of the wall by grouting wherever necessary. In addition, in view of its deteriorated and weak condition, the flint wall was encased in reinforced concrete tied to a continuous anchor beam landward of the parapet.

Considerable thought was given to the possibility of improving and stabilizing the foreshore by means of additional groynes and by altering or lengthening the existing ones. The effect of groynes upon the littoral drift is very much less pronounced and less obvious at Cromer than it is at Sheringham, but this is invariably the case where flat and predominantly sandy foreshores occur.

The long widely spaced groynes to the west of Cromer had built up sufficient beach material over the years to keep the sea away from the cliff, except during exceptional tides and storms, as can be seen from *Fig. 6*. On the other hand, the shorter groynes in front of the town were clearly ineffectual in retaining beach material under certain adverse storm conditions.

The fishermen are strongly opposed to groynes, and a decision on a recommendation that two new groynes should be constructed west of the town as a protection to a long length of sea wall and cliff (where no groynes existed) was deferred, but the engineers were allowed, albeit with many misgivings on the part of some of the fishermen, to lengthen two of the existing groynes, and they have since been authorized to build an entirely new one as a first instalment of the complete scheme.

The construction of the apron and encasement was straightforward and no special difficulty was encountered in carrying it out, except when a bed of a hard conglomerate resembling a weak concrete was penetrated by the sheet-piling at a depth of about 8 feet. This had to be dug out and broken up before the piles could be driven to full penetration.

Access to the beach for heavy plant was as easy at Cromer as it was difficult at Sheringham, and this greatly facilitated the execution of the tidal parts of the work. The apron and encasement of the flint wall were completed early in 1951 at a cost of £10 8*s*. 0*d*. per foot run for the apron, and £7 8*s*. 0*d*. per foot run for the encasement. The total cost of the work so far completed has amounted to £24,000.

The Contractors for the apron and encasement were Messrs W. G. C. Hammond Ltd, of Mablethorpe, Lincolnshire, and for the subsequent groyne work, Messrs May, Gurney & Co. Ltd, of Norwich.

The Suffolk Coast

Southwold

This pleasant little seaside town lies at an average level of 30 feet above mean sea level and is surrounded on three sides by marshland. The river Blyth flowing into the sea on the southern boundary forms Southwold Harbour, and Buss Creek on the northern boundary is reputed to be a former outfall of the river. The cliffs are composed of sand and gravel. The beach is of shingle and sand, underlain by sand to a very great depth.

The range of spring tides is 7 feet, the drift is on balance from north to south, and the interception of the drift material by the North Pier of the harbour has produced an extensive deposit of shingle which extends northward from the harbour for a distance of 3,000 feet. *Fig. 10* shows the accretion north of the harbour and the erosion south of the harbour at Walberswick. Material has been gathering since 1750, when piers were first constructed at the entrance to the harbour, and has extended further seaward since the North Pier was lengthened just before the 1939–45 war.

During periods of predominantly southerly winds, the direction of the drift is naturally reversed, and scour takes place northward of the northernmost groyne. Periods during which the littoral drift is on balance from south to north are fairly frequent, and on occasions last for several months.

In 1946, the Southwold defences consisted of twelve timber groynes in lengths ranging up to 200 feet, from 200 to 400 feet apart, and 4,000 feet of timber sheet-piled breastwork, encased in concrete for a distance of 1,800 feet south of the town pier. Some of these works dated back to 1898.

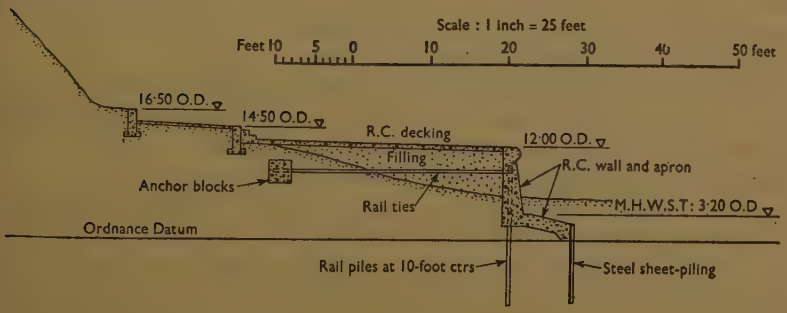
The crest of the shingle bank which separates the sea from the marshes at the north end of the town was found in places to be only 6 feet above

M.H.W.S.T., while the old breastwork which originally supported the seaward face of the bank was practically derelict. Most of the groynes and a large part of the breastwork at the centre of the frontage were also in a weak state, whilst the North Pier of the harbour, which originally contained a hearting of mass concrete partly deposited under water, was found on examination to be mostly hollow.

The condition of the defences in June 1946, when the Author was first consulted by the Borough Council, was clearly such as to give rise to concern for the safety of the town, and a scheme was prepared at an estimated cost of £30,000 for the construction of 750 feet of sea wall, five new groynes, and for repairing and lengthening nine of the old groynes.

The scheme was approved, but before work had commenced a severe storm in October 1947 scoured a gully against the face of the breastwork

Fig. 11



SOUTHWOLD, SUFFOLK.
NEW SEA WALL WITH CONCRETE APRON AND SHEET-PILED TOE

to a depth of up to 10 feet below original beach level. It revealed numerous gaps between the timber sheet-piles through which the shingle backing had escaped, and the scheme which had included a piled-and-anchored self-supporting reinforced-concrete encasement to the existing work was modified to include a steel sheet-piled apron and extended to cover a total length of frontage of 1,760 feet.

The scour also revealed traces of an old brick-and-concrete defence, of which no records existed. Since the debris was scattered about at depths up to 8 feet below original beach level, and lay in large lumps on the line of the steel sheet-piling, a considerable amount of deep excavation had to be carried out in open cut before the piles could be pitched and driven.

A typical section of the new sea wall is shown in *Fig. 11*. The wall was constructed partly as an encasement to the existing timber breastwork and partly over a frontage where no previous defence existed.

A piled-and-anchored form of construction was adopted as a safeguard against damage by undermining during construction, the design being very

similar to the one adopted under similar conditions at Hayling Island a few months previously.

Rail piles were driven at 10-foot centres and secured to anchor blocks by means of rail ties 30 feet in length. The piles were then encased in reinforced concrete and shingle-filling placed behind the wall. This completed the first stage of the work, giving full protection to the cliff. A row of steel sheet-piles 10 feet in length was then driven seaward of the wall, and the intervening space surfaced with reinforced concrete 18 inches thick, the mesh reinforcement of the apron serving as a continuous tie between the main piles and the sheet piles.

Finally the area between the wall and the cliff was surfaced with reinforced concrete at two levels, 6 inches and 4 inches in thickness.

By piling up excavated shingle in a mound seaward of the new work, it was possible to construct nearly all the new walls without interference from the tide. A horse with a drag-scoop was found to be more mobile than an excavator, in view of the confined conditions caused by the close spacing of the groynes, and this economical piece of " plant " paid for itself several times over and never broke down.

The new and reconditioned groynes in front of the town have given good results, the former more so than the latter, which have necessitated further maintenance from time to time. The three new isolated groynes, north of the town pier, which can be seen in the foreground in *Fig. 10*, have had to be strengthened from time to time since they were completed, on account of the wide variations in the level of the beach which have accompanied periodic reversals in the direction of the littoral drift. The northernmost of these three groynes is situated at the Borough boundary and, under the Coast Protection Act, Southwold Council are not permitted to go outside their boundaries and build works in other authorities' areas for their own protection. Other reasons for the difficulty which has been experienced in maintaining these three groynes have been the low level of the crest of the beach and the existence of marshland at the back, allowing shingle to be driven over the crest of the beach during exceptional tides and causing the inner ends of the groyne to be outflanked by the sea.

This situation has been met by providing a faggot breastwork along the back of the sea bank, but violent changes still occur in the profile of the beach and it is doubtful whether the situation will be fully under control until the Southwold defences are extended northward to the cliffs of Easton Bavants.

The new sea walls were completed in 1949 at an average cost of £23 per foot run including the decking and back parapet. The scheme as a whole cost £55,000 and the Contractors were Messrs R. Robinson & Co., of Westminster.

Aldeburgh

This small and ancient town of 2,600 inhabitants lies 12 miles south of

Southwold. In 1947, slow but progressive scour in front of the town was endangering the Brudenell wall, an anchored wall of steel sheet-piling, built in 1937 (without groynes) for the protection of property at the south end of the town. It was also becoming increasingly difficult to maintain a life-boat, the launching ways having to be progressively steepened.

A view of a large part of the town is seen in *Fig. 12*. The building standing on its own—landward of the small shelter in the immediate foreground—is the ancient Moot Hall, now the Council Offices. The Brudenell wall can be seen beyond the second coastguard tower with the new Stage 1 sea wall extending northward from it up to the tower.

South of the town, the land is composed of marshes through which the River Alde pursues its tortuous way to the sea at Shingle Street, 9 miles south of Aldeburgh. At its nearest point the river is only 300 feet from the sea and is separated from it by a shingle bank which was reinforced after the 1939-45 war by depositing a number of War Department defence blocks along the crest.

The Borough Council are responsible for the defences of the town as far as the south end of the Brudenell wall, and the East Suffolk Rivers Catchment Board—now the Norfolk and Suffolk Rivers Board—is responsible for the length of coastline beyond.

The Aldeburgh beach is composed of shingle with a fluctuating amount of sand in the vicinity of low water. The prevailing drift is from north to south, reversing to south to north periodically for periods of several months as in the case of Southwold.

There is no harbour at Aldeburgh and the fishing boats and life-boats are launched from the beach.

It was obvious that the Brudenell wall, being a static defence (the beach being mobile with a continuous natural tendency towards scour) would eventually collapse unless the beach also could be stabilized. This could only be achieved by means of groynes.

It will be seen from *Fig. 12* that, whereas there is a wide beach near the centre of the town, there is scarcely any beach left at the south end of the Brudenell wall, and the correct siting, spacing, and alignment of the groynes was a factor of considerable importance in the improvement and stabilization of the beach.

A scheme was prepared for dealing with the situation in two stages—Stage 1 comprising the immediate construction of seven groynes at the south end of the town, and Stage 2 the construction of twelve additional groynes to the north of them.

Conditions at that time were such that there seemed to be a reasonable chance of preventing further erosion by means of groynes alone, without having recourse to the construction of a sea wall or breastwork.

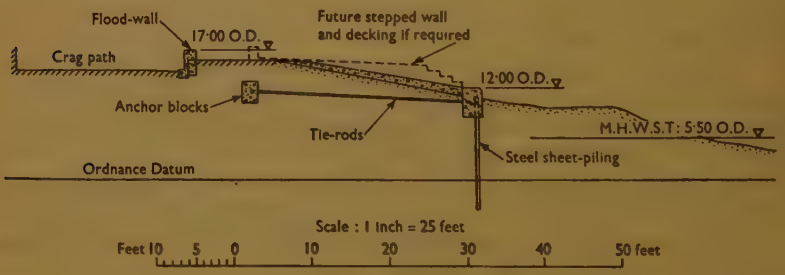
However, the construction of the Brudenell groynes coincided with a period of reverse drift, and the sea began to cut in towards the Crag Path north of the existing wall at such an alarming rate that a plan was approved

for the construction of 450 feet of sea wall in northward extension of the existing one as a matter of first priority.

The design for the sea wall comprised a steel sheet-piled and reinforced-concrete toe wall at a level of 6·50 feet above M.H.W.S., with a flood wall sited 30 feet to landward of it alongside the Crag Path founded on the anchor blocks with a top level 10·50 feet above M.H.W.S.

A typical section of the wall is shown in *Fig. 13*. It was designed in such a way as to allow for the future incorporation of a stepped concrete superstructure and a reinforced-concrete promenade decking in case it should be found that the toe wall, flood wall, and groynes were unable, *per se*, to protect the shingle bank landward of the toe wall against damage by scour. It was appreciated that a stepped wall and promenade

Fig. 13



ALDEBURGH, SUFFOLK.
SHEET-PILED WALL WITH CAPPING AND ANCHORS AND FLOOD WALL BEHIND

were undesirable features in a town where fishermen have to launch their boats from the beach and haul them up well clear of storm waves.

It is satisfactory, therefore, to record that there has been no tendency so far for the sea bank landward of the toe wall to be drawn down by storm waves, and the only places at which it was found necessary to supplement the toe wall with a stepped superstructure were at one of the coastguard towers and at the northern shelter, both of which were very near the crest of the beach.

The completion of Stages 1 and 2 of the work was followed by a further spell of strong southerly winds which caused serious inroads into the beach between the southernmost coastguard tower and the Moot Hall. This necessitated a further northward extension of the works including the provision of an additional 1,800 linear feet of sea wall and fourteen groynes. The works now terminate 300 feet north of the Moot Hall.

It says much for the reasonable and realistic outlook of the local fishermen and for the persuasive powers of the Mayor that there was no opposition at the public inquiry into the proposal to erect groynes at an average distance of 120 feet apart throughout the greater part of the frontage.

The fishermen have naturally benefited from the works but prejudice dies hard, and it is to be remembered that Aldeburgh has enjoyed in the past a happy immunity from sea walls, breastworks, and groynes, or any other obstructions to the free enjoyment and use of the beach.

No particular difficulties were encountered in the construction of the works. Special provision was made in the design of the three northernmost groynes to guard against the effect of a reverse drift—a wise provision—in view of the severe scour experienced north of the works in the winter of 1950-51.

The groynes were built partly of incised-and-creosoted Douglas fir and partly of home-grown elm depending upon what was available; they were sheet-piled. The driving was exceptionally hard and the sheet-piles had to be dug in and double-rail piles substituted for timber main piles in the ends of some of the groynes.

An infestation of Teredo spread northward in the summer of 1950 and affected the sheet-piles in fifteen of the groynes. Holes were drilled vertically into the tops of the sheet-piles, two per pile, to a depth of about 2 feet, filled with copper sulphate and sealed with corks. So far as can be determined this has checked the attack.

The works were completed in 1951 at a total cost of £64,000. The toe-wall and flood-wall defence shown in *Fig. 13* cost £15 10*s.* 0*d.* per foot run. The groynes cost £8 12*s.* 0*d.* per foot run.

The Contractors for the whole of the works were Messrs R. Robinson & Co., of Westminster.

The Paper is accompanied by six photographs and seven sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared, and by the following Appendix.

APPENDIX

The Effects of the February 1953 Storm

On the 31st January and the 1st February, 1953, an unprecedented storm surge, accompanied by winds of gale force, swept down the North Sea causing widespread flooding of tidal lands, grievous loss of life, and extensive damage to property.

As an indication of the exceptional height of the tide at the time of high water it may be stated that high water at Sheerness at 1.54 a.m. on Sunday morning, the 1st February, exceeded by 1 foot 10 inches the previous highest recorded tides of the 1st March, 1949, and the 12th February, 1938, and by 1 foot 11 inches the tide of the 29th November, 1897, which was stated to be the highest tide on record.

The extent to which the storm surge built up by stages on its way down the North Sea can be seen from column 4 of Table 1, which has been compiled from information obtained from various Harbour Authorities, River Boards, the Port of London Authority, the Admiralty Hydrographic Department, and resident engineers.

TABLE 1.—TIDE LEVELS TO O.D. NEWLYN

(1) Port	(2) Predicted level	(3) Actual level	(4) Storm surge : feet
Sunderland	+ 7·7	+11·3	3·6
Seaham Harbour	+ 6·9	+11·15	4·25
Immingham	+ 9·5	+15·25	5·75
Grimsby	+ 9·0	+14·9	5·9
King's Lynn	+10·7	+18·8	8·1
Great Yarmouth	+ 3·5	+10·75	7·25
Harwich	+ 5·68	+13·08	7·4
Southend	+ 8·8	+15·12	6·32
Whitstable	+ 7·7	+15·0	7·3
Sheerness	+ 8·3	+15·64	7·34
Chatham	+ 8·82	+15·12	6·3
Dover	+ 8·8	+15·0	6·2

Briefly, the effect of these abnormal conditions upon the sea defences described in the Paper was as follows :—

Lincolnshire

The reconstructed stepwork at the site of the Trusthorpe Breach shown in *Fig. 1* was undamaged but the parapet along the landward side of the promenade was overturned and the decking was undermined.

Light buildings just landward of the sea wall seem to have sustained little if any damage, notwithstanding the volume of water which came over the wall. The decking was 12 feet above M.H.W.S. and the top of the back parapet 14·5 feet above M.H.W.S.

Some of the clay backing to the new sea walls in the Huttoft area was scoured away but the walls were designed to be stable without support from the backing material and were practically undamaged. These walls were built to a level of 11·25 feet above M.H.W.S.

Lengths of old mass-concrete stepwork which had been encased with 9 inches of doubly reinforced concrete, as described in the Paper, were undamaged.

The new Sandilands wall and the Church Lane, Sutton, wall shown in *Fig. 2*, which were built to a level of 12 feet above M.H.W.S. and backed by a promenade decking and parapet were not only unaffected by the storm but gave complete protection to the sand-hills behind.

The back parapet and decking of the Chapel wall shown in *Fig. 3* were slightly damaged and a certain amount of backing material was scoured away, but the defences were not breached.

The old mass-concrete stepped walls which were built without any internal bond and without any back encasement to the contained filling were breached at many places.

The curved walls generally proved themselves very efficient in turning back the water and it would seem that walls of this type with coping at a level of 12 feet above M.H.W.S., backed by a reinforced-concrete promenade and parapet wall about 2 feet high (both considered as amenities at the time of construction), are proof against abnormal storm conditions.

Conditions at Trusthorpe and Sandilands were similar to the extent that beach level was approximately the same in each case whilst the top of the old stepped wall at Trusthorpe was at the same level as the top of the new curved wall at Sandilands. Nevertheless the sea came over the stepped wall at Trusthorpe with sufficient force to overturn the back parapet (level +25·0 O.D.) and cause serious flooding ; no such overtopping or flooding occurred at Sandilands.

Norfolk

None of the works described in the Paper sustained any damage, but the old walls and promenades at Cromer and Sheringham were badly knocked about. Most of the damage was caused by planes of weakness at the horizontal construction joints in these walls. The top of the wall was shattered and broke away at the joints exposing the base of the promenade slabs to the disruptive effects of wave action.

The cliffs at both places were badly scoured at heights up to 20 feet above M.H.W.S.

Suffolk

At Southwold the foot of the cliff was slightly scoured at one place and one or two beach huts on the promenade at a height of 11 feet above M.H.W.S. were slewed round. No damage was done to the new sea wall, which was built to a level of 8 feet 6 inches above M.H.W.S., with a wide decking and back parapet behind. The sea swept over the shingle bank to the north of the town and carried a large quantity of sand and shingle on to the marshes behind. The top of the bank was about 5 feet 6 inches above M.H.W.S

South of the town, between the termination of the new sea wall and the harbour, the sea breached the shingle bank and flooded the marshes. The top of this bank was approximately 10 feet 6 inches above M.H.W.S.

At the harbour where the breach is very wide the houses on the seaward side of the road were undamaged, although the road at this point is only 7 feet 6 inches above M.H.W.S.

At Aldeburgh the sea flowed over the crest of the beach north of the sea wall and over the new defences in front of the town causing some inundation of low-lying parts of the town but no appreciable damage. The top of the sea wall is 6 feet 6 inches above M.H.W.S. and the top of the flood wall 11 feet 6 inches above M.H.W.S. The intervening space, 25 to 40 feet wide, consists of shingle which was pushed up level with the top of the flood wall.

A short length of stepped wall with a curved parapet similar to the Lincolnshire walls which was built just south of the Moot Hall gave full protection to a shelter 6 feet behind it and very little shingle was thrown over the top. The top of this wall was 12 feet above M.H.W.S.

South of the town the whole shingle bank was rolled back into the marshes leaving the groynes unsupported and a breach was formed between the town and the river wall. The top of this bank was at a level of 8 feet to 12 feet above M.H.W.S. before the storm.

CORRESPONDENCE

on Papers published in Proceedings, Part II, February 1953

Maritime Paper No. 21

" Some Designs for Flexible Fenders " †

by

Donald Hamish Little, B.Sc.(Eng.), A.M.I.C.E.

Correspondence

Mr J. A. Williams stated that the Author seemed to have had rather mixed results with the type of structure depicted by *Figs 11* and *12*, although in his oral introduction he had briefly referred to some further developments which had been made.

Mr Williams was of the opinion that the novel design of kinetic-energy-absorbing fender, described by Mr Ridehalgh and illustrated in *Figs 25*, offered a number of advantages over swinging fenders, and he suggested that, even with the latter, some of the disadvantages experienced by the Author could, to some extent, be mitigated.

For instance, the vexed question of longitudinal blows which, as the Author had remarked, were often ignored altogether, might be tackled by connecting all the swinging blocks together laterally, so that when one received a glancing blow the resistance of a number of blocks was mobilized. That, of course, pre-supposed adequate lateral clearance between fenders and supporting structure—a feature which, in the later designs described in the Paper, had been deliberately avoided. Mr Williams submitted that the ideal aim was the provision of ample lateral " swinging room," rather than the confining of movement to slots normal to the face of the berth. The inter-connexion of the blocks might also help to damp out the effect of waves. Those advantages could only be gained, however, at the cost of making provision for somewhat obtuse secondary stresses in the blocks themselves, the suspenders, and the supporting structure. The maximum

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† Proc. Instn Civ. Engrs, Part II, vol. 2, p. 42 (Feb. 1953).

direct force which could be developed in the suspenders was similarly troublesome to evaluate, and model-analysis was no doubt desirable.

He suggested that the following improvements could be effected in the actual shape of the blocks :—

(*a*) The working faces of the blocks could be widened out in plan and the facing timberwork dressed, roughly, to curves, both horizontally and vertically. That would (1) afford more cover, per block, to the face of the berth ; (2) tend to make the fenders " ride " glancing blows ; and (3) tend to concentrate the blow at the neutral axis of the block.

(*b*) More " balance " with, probably, a reduction in secondary stresses, would be achieved with a comparatively wide and shallow block, rather than one which was narrow and deep (see *Figs 11* and *12*).

Mr Williams noted that difficulty had been experienced in tightening the turnbuckles in the suspension gear (see p. 58). He had recently contemplated a similar arrangement for a tentative design—the suspenders being chains ; had the design been adopted, no doubt the same trouble would have been experienced. Incidentally, it had been proposed to incorporate a locking device to prevent the couplings slackening back under operational conditions.

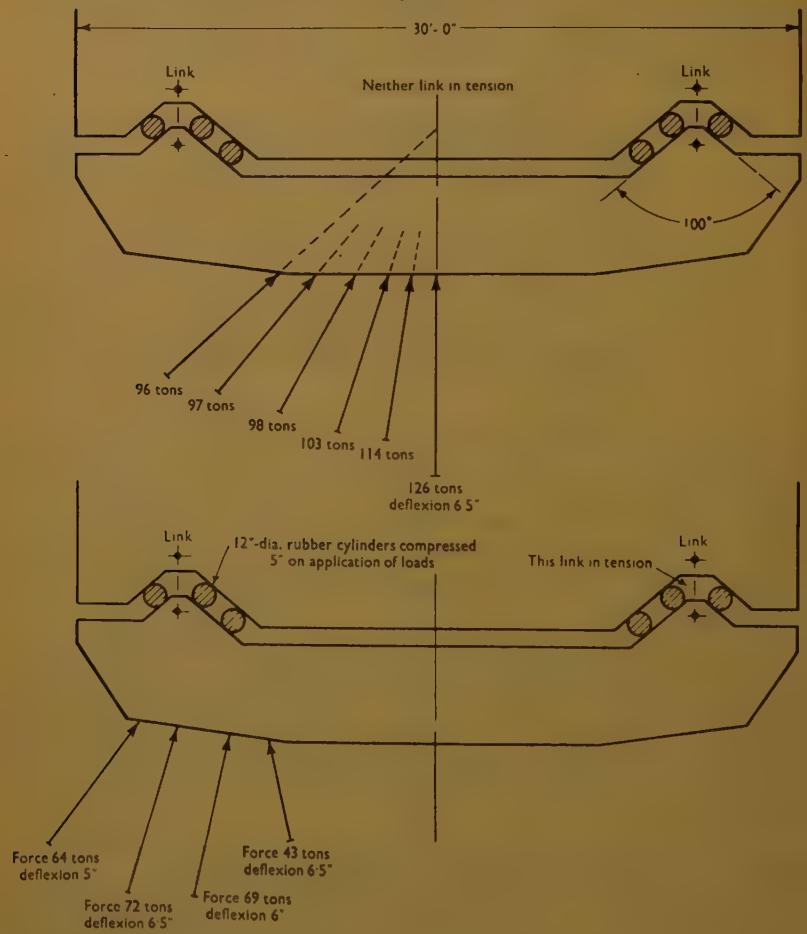
In that same tentative design, which catered for vessels of about 50,000 tons displacement, the intention had been to provide a swinging fender, with enlarged head, in each bent of the berth so that a very nearly continuous flexible-fendered face would be presented to the vessel. Each individual block was to be capable of absorbing 50 per cent of the assumed blow with (when struck at 90 degrees to the berth) an 18-inch rise and a 4-foot-6-inch travel.

Mr D. C. McCormick stated that the scheme referred to by Mr R. B. Kirwan in the oral discussion had been further developed.

The arrangement was shown diagrammatically in *Figs 38* ; the two wedges protruded from the back of an " apron " of concrete which either hung in front of the jetty or stood on a group of fender piles. Two strong tension links prevented the apron moving away from the jetty under an eccentric load and brought more of the rubber cylinders into use. The Figures showed the deflexions under a selection of different forces sufficient to compress some of the rubber cylinders by 5 inches, but the ultimate capacity of the group was considered to be well beyond those figures.

Mr A. M. Robertson was particularly interested in the Author's observations on gravity fenders since he personally had refrained from using them at exposed sites ; he had seen some at a Scottish loch pulsate considerably when there was extremely little wave action, indicating that they would tend to create trouble by excessive wear, and throw up water if used at exposed sites.

Figs 38



The design of simple fendering and dolphins appeared to have been retarded by the refusal of some engineers to accept the following easily proved and long-known facts :—

(*a*) The work absorbed by a member in bending was directly proportional to the length of the member, irrespective of whether it was simply supported or one of the unbraced legs of a dolphin.

(*b*) Provided the shear stress was not exceeded the work absorbed by a fender member in bending was independent of the location of the impact.

(*c*) The greater the deflexion of a fender, the more it would be relieved by its neighbours.

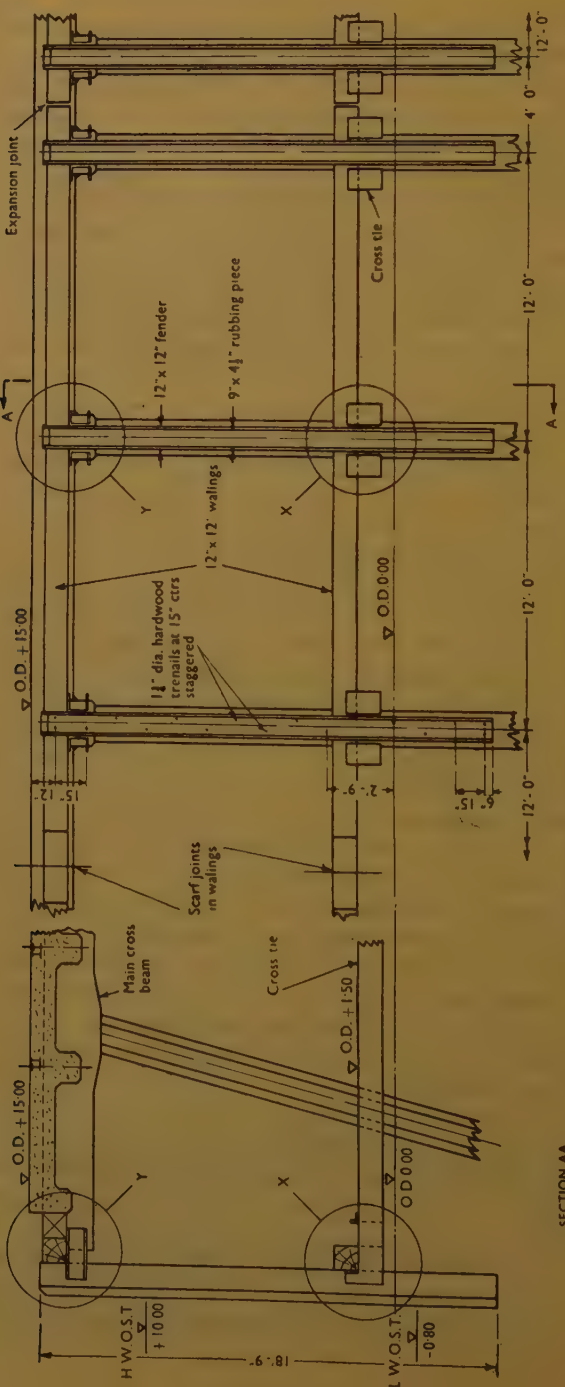
The Admiralty had demonstrated by numerous examples that it was practical and economical to base their designs on the above facts. Nevertheless, it was still common to see timber lead-in jetties and dolphins braced down much too far and fender piles supported at close intervals.

Fender design had, in Mr Robertson's view, also suffered from a lack of facts about the timber known as greenheart. That natural material was not just a little stronger than " softwood," but had more than twice the strength of Douglas fir in bending, and could absorb more than three times as much work for practically the same cost as pressure-creosoted Douglas fir. Furthermore, greenheart was not weakened by knots and faults as were many other timbers, and from observations carried out over the past 20 years, on both new and old greenheart, the latter obtained from structures more than 80 years old, it seemed to be the most consistent and fault-free of the timbers in use today. In recent bending tests on greenheart to check published figures, it had been found that three specimens having cross-sections of 2 inches by 2 inches broke at about 23,000 lb. per square inch. That was two-thirds of the yield-point stress and one-third of the breaking stress of mild steel, and twice the crushing resistance of the best factory-made concrete.

When stressed to only 11,700 lb. per square inch, single 60-foot greenheart logs, of 14-inch-by-14-inch and 16-inch-by-16-inch section, would absorb 146 and 192 inch-tons respectively, but owing to the favourable shape of the " work diagram," they would absorb approximately 730 and 960 inch-tons respectively before breaking. It was therefore very difficult to decide just how much work greenheart fendering could be relied upon to absorb, but its capacity was obviously very high and superior to any other fender available at the same cost; consequently very great advantages and economies would result from tests on sections of greenheart fendering, backed by systematic tests on full logs as imported, to determine the effect of repeated loading in the region of the breaking load. Mr Robertson was thinking in terms of fendering for general-purpose wharfs and jetties, and believed that vertical greenheart fenders, suitably grouped by horizontal walings, and supported at deck level by long longitudinal greenheart logs arranged to spring under or over the deck, and supported at their ends with bonded-rubber units, would provide the most economical and satisfactory form of fendering. A greenheart deck " spring " was much cheaper than rubber units at each vertical.

It would be interesting to have the Author's views on the merits of vertical and battered fender piles. Battered piles would appear to be capable of rocking wall-sided vessels and making them absorb work; they would also reduce the deck reaction, and in consequence lead to cheaper main structures.

Mr A. H. Robinson, referring to design No. 1, observed that a jetty with similar fenders had been constructed at Singapore about 1938. That jetty had fenders, illustrated in *Figs 39* (*a*), (*b*), and (*c*), which

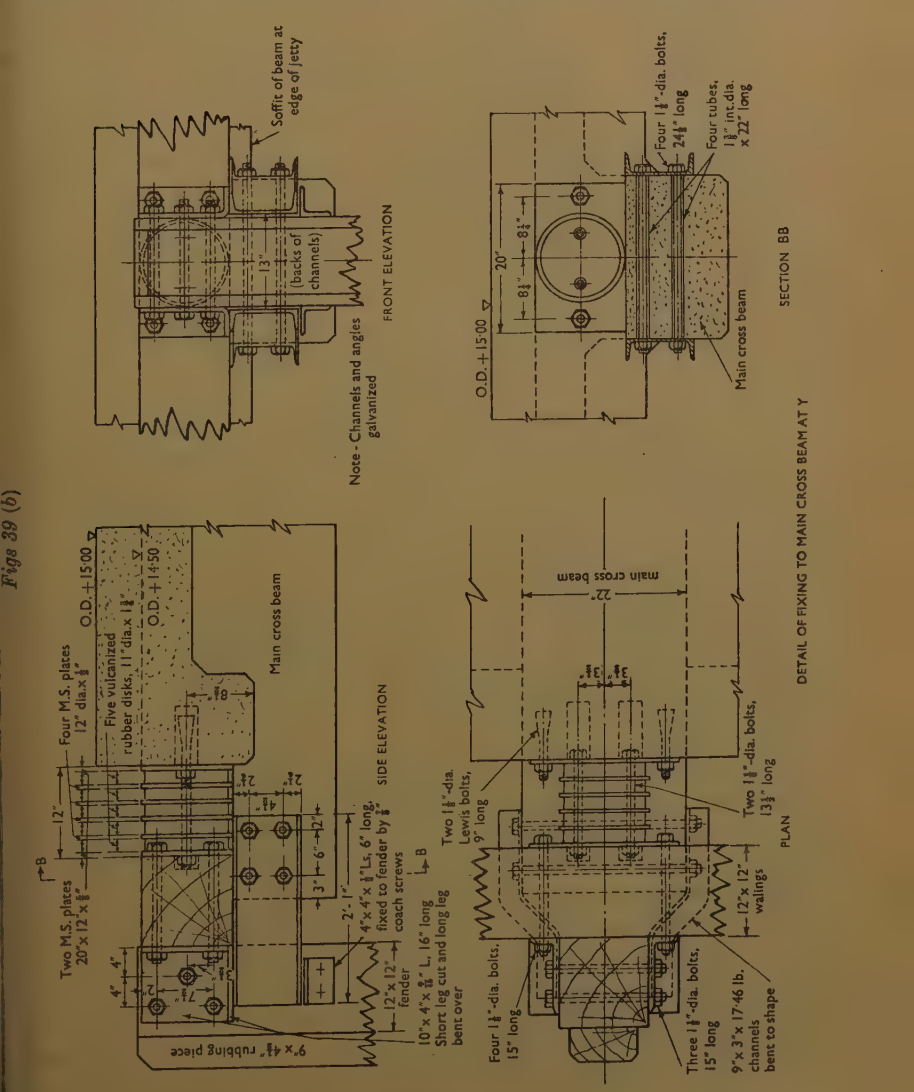


Figs 39 (a)

GENERAL DETAILS OF JETTY HEAD

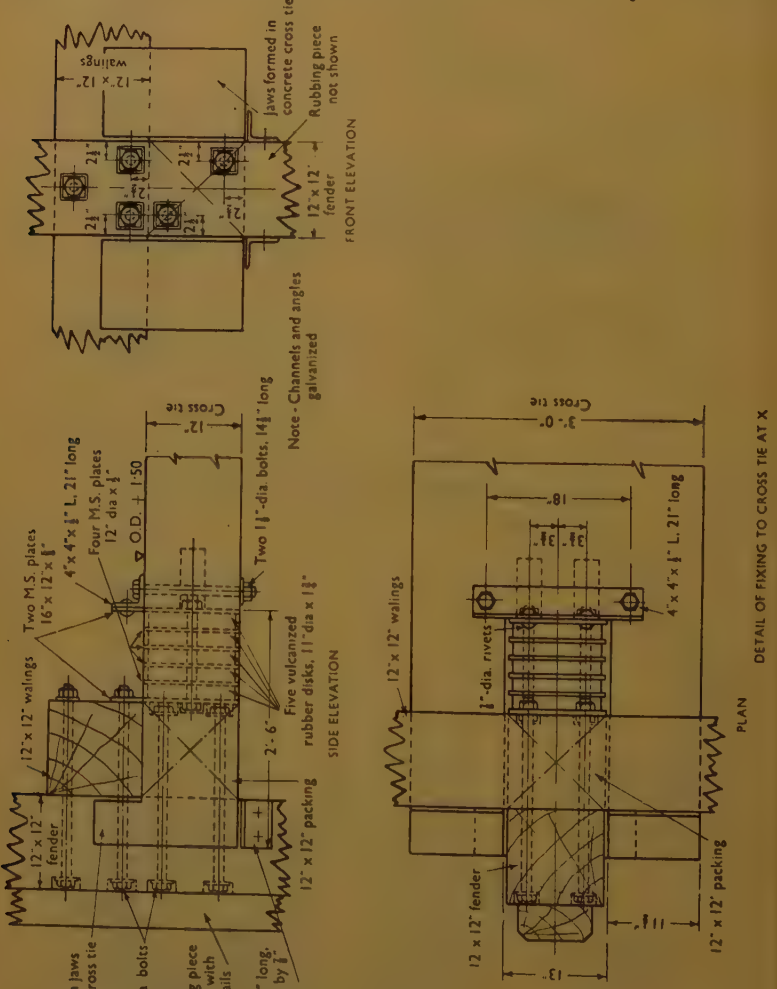
cantilevered down below the low-water line and were fixed by rubber buffers at high and low level, as distinct from driven fender piles with buffers at high level only as in design No. 1. It also had 12-inch-by-12-inch horizontal walings at high and low level, which would transmit a glancing blow along the line of fenders.

So far as could be ascertained, the buffers at present fitted were the original ones. The high-level ones were as good today as the day they



were fitted, even though they had been exposed to the tropical sun and rain for about 14 years, but the rubber at the low level, which was between high- and low-water marks, had hardened.

The fendering had proved itself more robust than design No. 1, mainly because of its capacity to transmit glancing blows through the heavy horizontal walings.


Jaws formed in concrete cross tie
Rubbing piece not shown
12" x 12" fender
FRONT ELEVATION
Note - Channels and angles galvanized
Two 1¼"-dia bolts, 14¼" long
Cross tie
12"
Two M.S. plates 16" x 12" x ½"
4" x 4" x ½" L, 21" long
Four M.S. plates 12" dia x ½"
O.D. ± 1·50
12" x 12" walings
12" x 12" fender
Five vulcanized rubber disks, 11" dia x 1½"
2'-6"
12" x 12" packing
SIDE ELEVATION
Stops cast on jaws of concrete cross tie
1¼"-dia bolts
9" x 4½" rubbing piece fixed to fender with hardwood trenails
4" x 4" x ½" Ls, 6" long, fixed to fender by ½" coach screws
Cross tie
3'-0"
12" x 12" walings
4" x 4" x ½" L, 21" long
½"-dia rivets
12 x 12" fender
12" x 12" packing
PLAN
DETAIL OF FIXING TO CROSS TIE AT X

Figs. 39 (c)

Additional protection for the jetty was now provided by the use of catamarans which might well be provided with advantage for all jetties. It would probably help greatly if catamarans came to be regarded as an integral part of the structure and not merely as a desirable appendage as they were usually looked upon at present.

Mr H. S. Waddington remarked that many examples were given in the Paper of fendering consisting of a row of isolated piles driven into the sea or river bottom in front of a wharf or jetty and held only at cope level. Would the Author say how those isolated piles were standing up in use, particularly with regard to glancing longitudinal blows ?

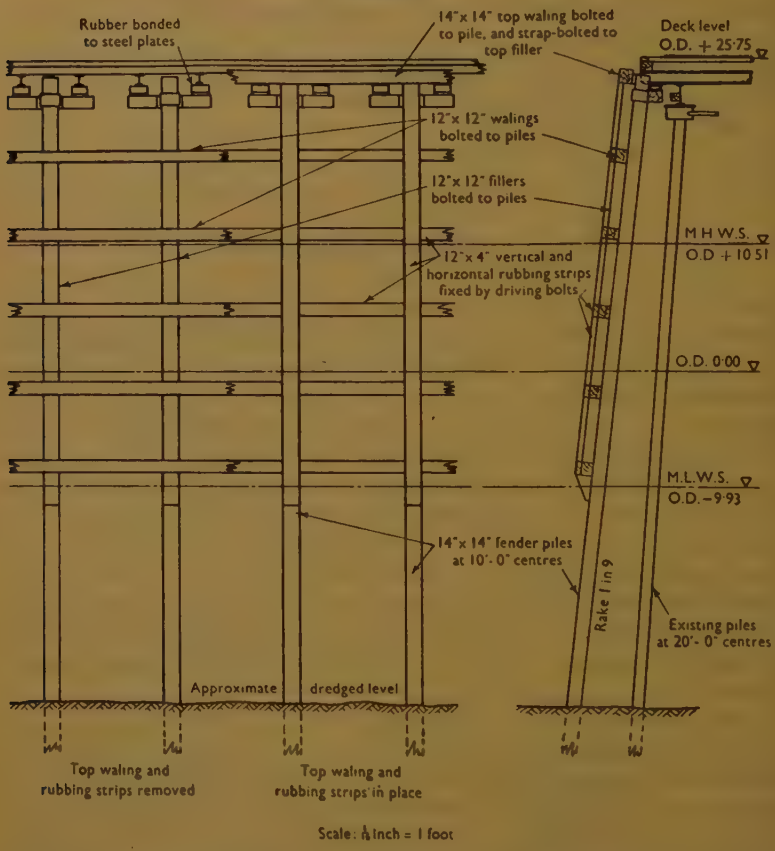
Design No. 19, with its timber piles unsupported for 50 feet, was delightfully simple—almost too simple to be true, especially in view of the sizeable vessels (4,000 tons) expected to use the berth. The arrangement adopted with piles at 10-foot centres was fundamentally the same as in the superseded design No. 17 at that jetty, except that the old piles were presumably at 20-foot centres. It was found that timber rubbing pieces had been torn off the earlier fendering system ; was the greater longitudinal " give " in the new design preventing a recurrence of that trouble ? On p. 76 of the Paper it was suggested that the effect of the impact load was often near the deck, and that was corroborated in an earlier Paper.[1] To guard against the possibility of a flared bow making direct contact with the deck of the jetty and maybe, if under way, catching the side of a fender, a horizontal fender would appear desirable at about cope level. In order to deal with glancing blows at lower levels, further horizontals could be provided.

Had the Author considered any scheme, somewhat on the lines of *Figs 40*, as a variation to design No. 19 ? The diameter of the rubber buffers would be less than in the Author's design because the stiffness of the timberwork would bring more buffers into play. Fendering with that type of face (verticals at about 10 feet and horizontals at 5-, 6-, or 7-foot centres) had been common practice for many years on timber quays. Although it might be thought that small craft would run the risk of being caught under the horizontals on a rising tide, experience indicated that no such difficulty was found with vessels down to the size of tugs and barges. Whether such an arrangement would be suitable for the site for which design No. 19 was prepared would depend on the type of craft and other local conditions. The scheme outlined in the sketch would be more expensive in first cost than the Author's design, but would give full protection to the jetty against ships' blows from all directions and at all points, would spread the impact load over a longer length of the jetty, and would probably not require so much or such constant maintenance attention.

Mr H. W. Stephenson, referring to the berthing velocity of ½ foot per second recommended on p. 75 of the Paper, observed that at one wharf

[1] A. W. Rooke, " Improvements in Jetty Design, with Special Reference to Systems of Fendering." Maritime Paper No. 14, Instn Civ. Engrs, 1949.

Figs 40


Rubber bonded to steel plates
14"x 14" top waling bolted to pile, and strap-bolted to top filler
Deck level O.D. + 25·75
12"x 12" walings bolted to piles
12"x 12" fillers bolted to piles
M.H.W.S. O.D + 10·51
12"x 4" vertical and horizontal rubbing strips fixed by driving bolts
O.D. 0·00
M.L.W.S. O.D. −9·93
14"x 14" fender piles at 10'-0" centres
Rake 1 in 9
Existing piles at 20'-0" centres
Approximate dredged level
Top waling and rubbing strips removed
Top waling and rubbing strips in place
Scale: ⅛ inch = 1 foot

where flexible fenders had been installed, observed deflexions inferred berthing velocities on not infrequent occasions as high as 1¼ feet per second for ships of 10,000 tons. It seemed clear that, had the design berthing velocity been ½ foot per second, the fenders' life would have been uneconomically short. The maximum absorption of elastic energy per fender was 3,100 inch-tons, the corresponding deflexion at deck level being 4 feet 6 inches.

The fenders consisted of three rows of H-piles framed together to spread a point impact but unbraced in any direction, with a pipe pile cluster at each end (see *Fig. 41* (*a*)). The piles were of multi-section (*Figs 41* (*b*)) for the most efficient use of elastic energy. Yield point was reached throughout the piles at full deflexion, when the fenders touched the cantilever section of the wharf (see *Fig. 41* (*c*)). For a catastrophic impact, the cantilever section was designed to crumple as the fender was deflected past yield point,

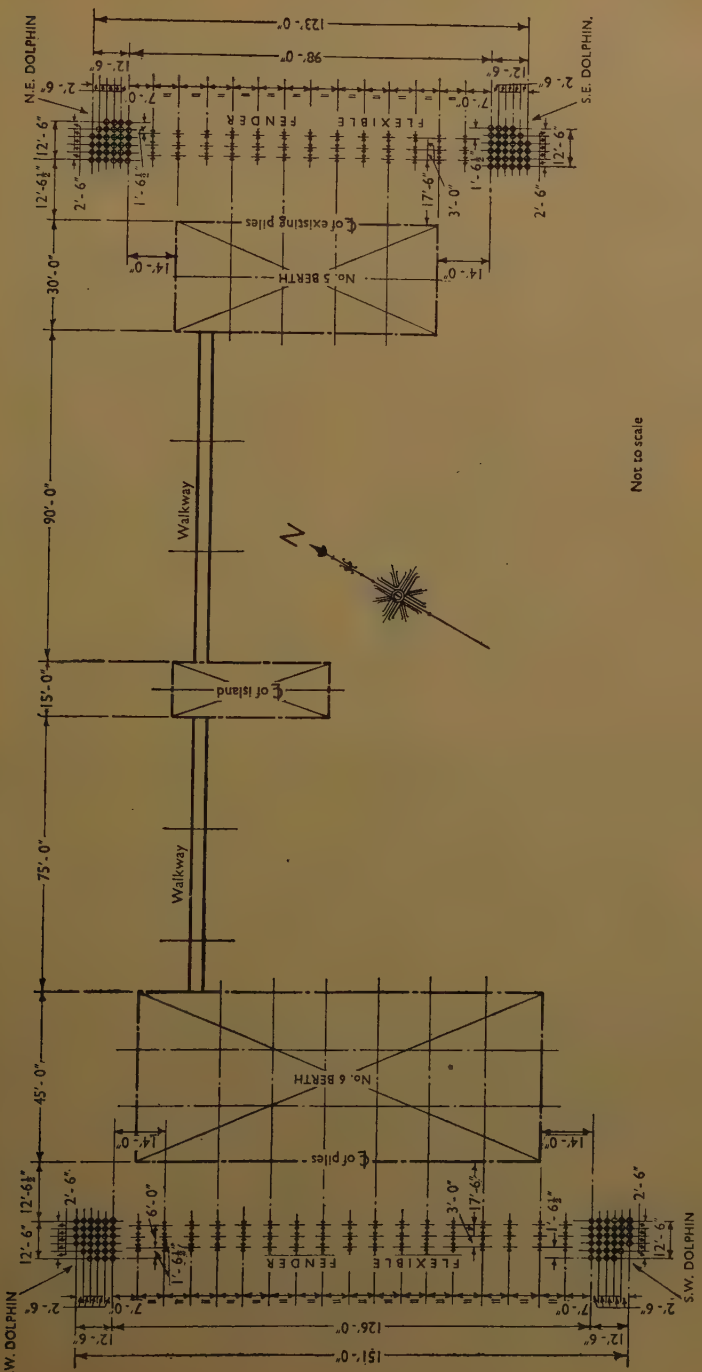
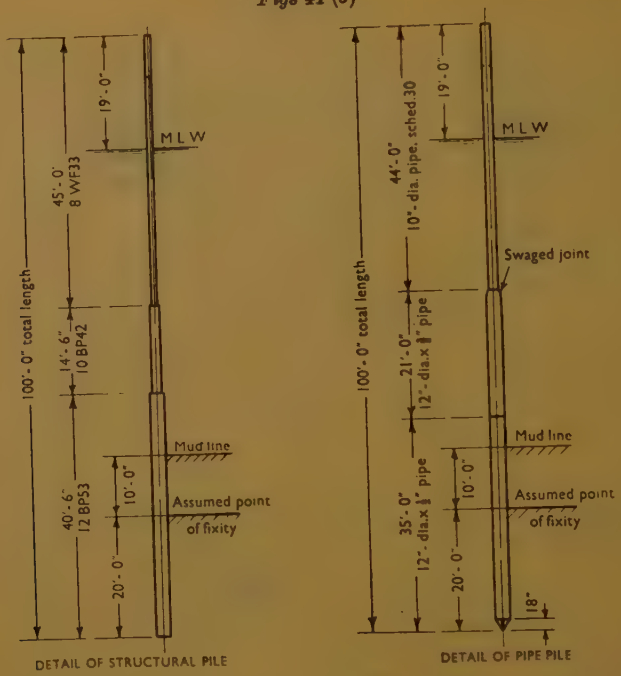


Fig. 41 (a)

GENERAL ARRANGEMENT OF WHARF AND FENDERS

Figs 41 (b)

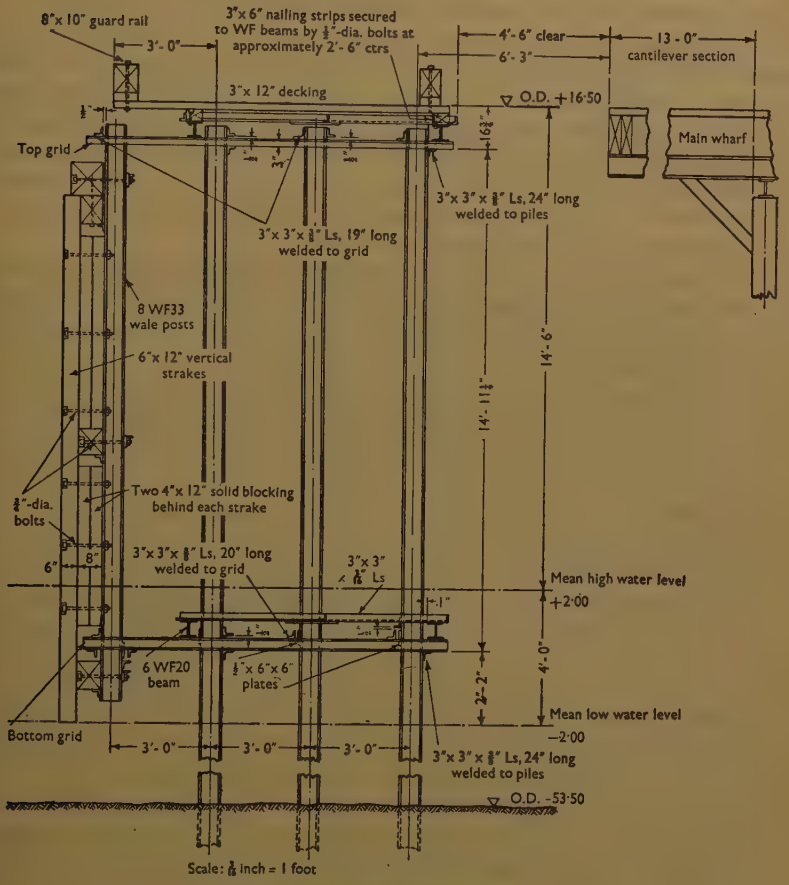


and energy continued to be absorbed non-elastically until the fende contacted the main wharf structure. The non-elastic energy would b considerably in excess of the elastic.

The cost of the two fenders was £10 per inch-ton of elastic energy, fo 20 inch-tons per linear foot of wharf, or £200 per linear foot of wharf It was not possible, owing to the weakened condition of the main wha structure, to allow any lateral load to be transmitted to it by the fender however cushioned, and that factor contributed to the high cost. Th fenders, however, in effect provided extra wharf space, and suggested th possibility, in suitable circumstances, of a flexible wharf.

Shortly after the completion of the wharf, a ship unberthing had struc a corner dolphin a diagonal blow. There had been no apparent set in th longitudinal direction but the connexions between dolphin and wharf ha sheared, and the dolphin had sustained a permanent set, in the transver direction, of approximately 6 feet. When the timber framing (bolte 18-inch-by-12-inch sections) of the dolphin had been removed duri repairs, the piles had sprung back to within 2 feet of their original positio revealing a valuable source of energy absorption in the deformation of t timber framing. It was estimated that the energy of the impact had be 3,000 inch-tons, including non-elastic deformation.

Fig. 41 (c)



TYPICAL SECTION OF FENDER

The Author, in reply, stated that heavy isolated free-swinging fenders, wider in plan, as suggested by Mr Williams, were incorporated in a jetty now under construction, and a series of smaller continuous gravity fenders were included in another jetty which it was hoped would be built soon. Designs on both those lines, however, were difficult to evaluate completely and it would be premature to discuss them until some experience of their actual behaviour in use was available.

The arrangement shown by Mr McCormick in *Figs 38* was an ingenious method of taking blows at various angles by rubber, mainly in compression all the time. But the movement of the fender unit proper at 6·5 inches was not very great and, as a consequence, although the thrust of 126 tons was quite high, the kinetic energy absorbed was hardly 400 inch-tons.

Mr Robertson's reference to greenheart was particularly important. Even greenheart could be severely attacked by Teredo in some waters, but where that was known to be impossible the Author agreed that greenheart fender piles with some flexibility provided at deck level should produce a simple, effective, and relatively cheap solution. He had not had much experience with battered fender piles except that those in design No. 17 had given no indication that they would have failed sooner had they been vertical. It was doubtful if the reduction in deck reaction with battered fender piles would be really significant, and on the whole the Author saw no positive advantage in them.

Mr Robinson's confirmation as to the life of rubber itself was valuable but, although that fendering system had lasted well, it was a design which necessitated tidal bracing—almost at low-water level—and that in itself would add considerably to initial costs.

Mr Waddington's proposal in *Figs 40* was thought to be an improvement, but general Admiralty practice was to avoid longitudinal fendering wherever possible, and in the case of design No. 19 it was also necessary to keep first costs as low as possible. That design had not yet been built and if, when it was, it showed signs of weakness some strengthening with longitudinal fendering might have to be accepted.

Mr Stephenson's example was very interesting. The Author agreed that when actual berthing velocities were known, they should be adopted in place of the arbitary figures of $\frac{1}{2}$ foot per second. But in the case quoted by Mr Stephenson it seemed that head-on collisions embodying the full weight of the ship, rather than half of it, were involved. Those were very severe conditions indeed, and if it was not possible, or not economical from a shipping viewpoint, to insist upon more careful berthing of the ships, then the costly remedy of fendering at £200 per linear foot had to be accepted. It would be interesting to know if the use of greenheart instead of steel would have been practicable and cheaper. Deliberately to design for deflexions of 4 feet 6 inches was a bold attitude but, in the Author's view, a very proper one, and was well confirmed by the subsequently recorded deflexion of 6 feet in one of the dolphins. It was difficult to imagine, however, that movements of that order could ever be tolerated in a wharf structure as a whole.

Railway Paper No. 47

" Some Major Problems in Railway Civil Engineering Maintenance " †

by

Alfred Henry Cantrell, B.Sc.(Eng.), M.I.C.E.

Correspondence

Mr W. T. Wilks was of the opinion that the civil engineer in, say, the chemical or petroleum industries needed a larger measure of theoretical training, practical experience, and common sense than, as suggested by the Author, the engineer responsible for maintenance on the railways. In those industries, the engineer had to deal frequently with gases which could could not be seen, heard, or controlled as could, for example, trains.

Referring to the re-flooring of Charing Cross Bridge (see p. 120), Mr Wilks expressed surprise that the steel deck plates had been riveted to the rail bearers and cross-girders. Despite the deflexion in the floor of a bridge, the use of electric welding was not precluded if the right design and technique were used. There existed many bridges in which the deck-plate rivets had worked loose—allowing water to percolate through the joints. He suggested that electric welding would have reduced considerably the time of reconstruction of that particular bridge.

Mr Wilks could not understand the reason for the renewal of the bottom flange of the cross-girder in such heavy sections, and costly method of construction, of the bridge over the River Thames at Staines (*Figs 10*). The bending moment at the ends of that simply supported girder was very small and was provided for in the material which remained in the vertical leg of the existing flange angle and a portion of the web plate. The welding of the bottom flange of the cross-girder to the main girder might overcome the difficulty of corrosion but might well introduce secondary stresses in the latter to its ultimate detriment.

The Author should have given a word of warning to the engineer with limited knowledge of electric welding on its indiscriminate use for repairs to bridge work, since the expansion, contraction, and distortion induced in the riveted work might produce excess stresses which could be serious; also the danger of notch brittleness had to be borne in mind. Before 1939, German engineers had strengthened some of their old riveted bridges by welding, but abandoned the practice after a short time when it had been discovered what was happening.

One of the most important bridge works undertaken by the old Great Western Railway in the past fourteen years was the reconstruction of the

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† Proc. Instn Civ. Engrs, Part II, vol. 2, p. 106 (Feb. 1953).

Kidwelly Viaduct in South Wales. The preparation of the design, erection scheme, and contract had been entrusted to Mr Wilks. That structure carried the up and down South Wales main lines over the River Gwendraeth Fach, at a distance of 234 miles from London. The original viaduct, built about 1852, had been in timber, but had been replaced in 1894 by the wrought-iron structure which had also had to be renewed.

The viaduct was made up of four 68-foot spans ; the sub-structure consisted of two stone abutments and three piers in the river bed. Each of the piers comprised three wrought-iron cylinders 6 feet in diameter, set 15 feet $1\frac{1}{2}$ inches apart, filled with concrete, and braced transversely at the top with wrought-iron girders to act as one unit. The old superstructure which had been replaced was of the plate-girder through-bridge type, having one line of centre girders and two lines of outside girders which carried cross-girders 4 feet apart, supporting timber decking on which was laid the ballast and permanent way.

The cylinder piers were found to be in such good condition that small repairs only were necessary to put them in order, and those had been carried out by (electric) welding patch-plates to the affected parts. The re-use of the piers had decided the type of new superstructure, which was similar to the old, except that steel deck plates riveted to rolled-steel-beam rail bearers were used, instead of timber decking. There was nothing of unusual interest in that construction, apart from the fact that the viaduct was set out on a curve. The interesting work was the erection scheme which in principle was to renew, first of all, the half of the viaduct carrying the up line, while keeping the down line open for all traffic.

The first operation had been to slew the down line away from the centre girders, in order to permit the cutting away of a portion of the timber decking carrying that line, to give access to the ends of the cross-girder, and also erect in the space so provided a temporary erection girder alongside an existing centre girder, and, from the former, to carry the ends of the cross-girders by means of suspension bolts. The existing centre girder, relieved of all load, was then removed, also the outside girder, timbers, and cross-girders supporting the up line. With that portion of the span free, the new centre and outside girders, cross-girders, rail bearers, and steel deck plates were then fixed.

Those operations constituted all the work necessary to complete the erection of steelwork for carrying the up line for one span. Work was first begun on the span nearest Kidwelly station, and on completion of that, the next span was fixed, still for the up line, until the four were finished. Afterwards, the permanent way of the up line was temporarily re-layed and the traffic diverted over it, while the down line was closed for reconstruction.

It should be noted that before the erection girder could be moved to another span, the weight of the existing cross-girders had to be released from it and transferred to the new centre girder.

In fixing the steelwork for the down-line section, the erection girder was unnecessary, since there were no cross-girders to be held up at that stage of operations. Consequently, the speed of erection was accelerated, and two spans were fixed at one period of occupation, which usually was from Saturday night to Sunday noon.

The erection girder used on that job had been built originally for the reconstruction of Saltash Viaduct; it had, however, to be lengthened for the work at Kidwelly. Provision had to be made for supporting the end of the girder from the old structure, and that support comprised double 17-inch-by-4-inch rolled steel cross-channels slung underneath the floor and hung by heavy flat steel bars from cross-head-channels resting on top of the existing outside and centre girders.

The heaviest piece to be lifted was the 27-ton erection girder, and two of the Company's cranes, each of 15 tons capacity, were requisitioned for the lifting of that and all main girders. Those cranes occupied the up line during its reconstruction and vice versa for the down line, thus keeping one line free at all times.

Each of the major operations described above had had to be worked out in detail to ensure that the cranes could manoeuvre in restricted spaces, and stand in predetermined positions, without unduly overloading the new and old structures. Also, it had been necessary to make an accurate estimate of the time taken for each operation, so that a time schedule could be compiled, acceptable to the Traffic Department, enabling that department to give permission for complete occupation of the lines while the principal and heavy members of the viaduct were lifted into position. Further, the co-operation of the steelwork fabricators was essential, to obtain delivery of steelwork at specified times, and in the correct order.

Although there was approximately only 15 feet clearance between low-water level and the soffit of the girders, the scheme was very much cheaper than the alternative one of erecting temporary staging from the river bed, on which to carry the new superstructure. Also, the limited headroom for driving temporary piles for carrying the temporary staging would have presented a problem which could not have been easily solved without much interference with traffic.

The Author, in reply, stated that welding the deck plates of Charing Cross Bridge had been considered in the initial stages of preparing the scheme, but it had been decided that a quicker and more convenient job could be carried out by riveting. That method, when carried out efficiently, should not lead to rivets becoming loose although he was aware there were some cases in which that had happened. In the design of the new floor at Charing Cross, dished plates had been used, with drainage space along the centre-lines so that there should be no accumulation of water near the rivet heads. The floor was fabricated in the shop and erected in large panels at the site, and therefore any saving in time that might be claimed for welding, as compared with riveting, would not be material since there

would be no effect on the time required in erection at the site. Little difficulty had been experienced in maintaining riveted floors on the large number of old bridges.

The reason for the renewal of the bottom flange of the cross-girders of the bridge at Staines, in the manner shown, was that severe corrosion had taken place at the cross-girder ends and it was therefore necessary to renew the bearings on the main-girder flanges. To avoid damage to the main girder, the lower portion of the cross-girder was burnt out slightly above the level of the outstanding leg of the lower angle, and it was cheaper and more satisfactory to provide a substantial bearing plate to fill up the space available and give good distribution on the main girder, than would have been the case with other methods. The whole of the work had been carefully carried out and planned to ensure that secondary stresses were not introduced by the welding. Detailed examination with magnifying glasses had not revealed any cracks in the welds although some of them had been made more than 2 years previously.

The Author agreed that a word of warning on the indiscriminate use of electric welding repairs to bridges might have been given, but the care required and the serious results which might occur after a badly conceived repair were well known and therefore the omission was not serious.

The experience gained by the Germans before 1939 was also gained in Great Britain, and before the war, repairs by welding on bridges—on the former Southern Railway—were carried out, but great care had been used in their design. They had proved quite satisfactory but tests were made and stresses measured before schemes were prepared.

CORRIGENDA

Proceedings, Part II, February 1953—

Maritime Paper No. 21 (Little), Discussion by R. C. Harvey, p. 100, line 12.—*For* "3·3" *read* "2·3."

Paper No. 5897 (Crowther). *Fig. 8 (a)*, facing p. 205.—*For* "WALL AND" *read* "WALLAND."

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